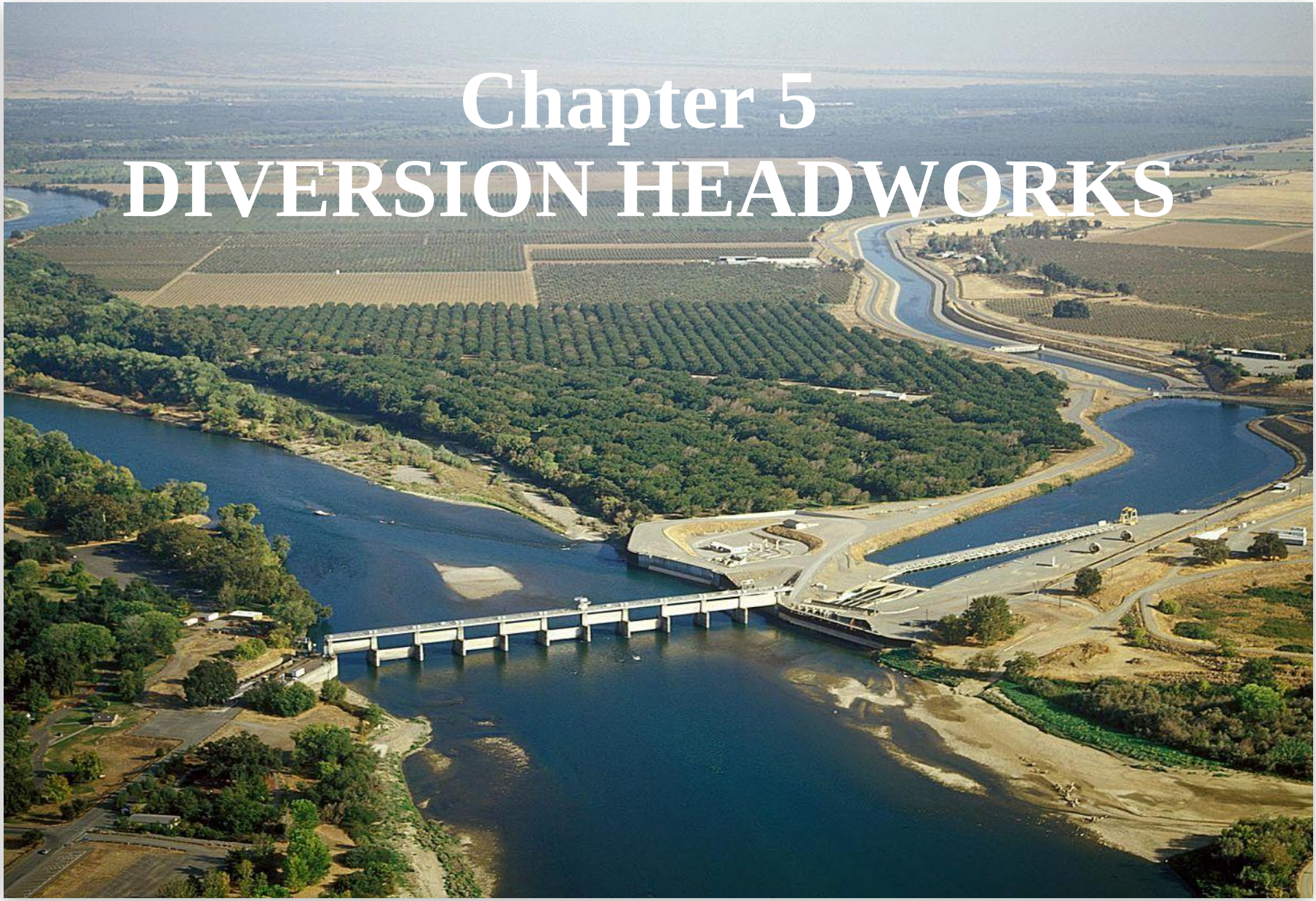
Chapter 5
DIVERSION HEADWORKS



Diversion Headwork

The works which are constructed at the head of the canal in order to divert the river water toward the canal so as to ensure a regulated continuous supply mostly silt free water with certain minimum head into the canal are known as diversion headworks.

Functions of Diversion Headworks

The major functions of diversion headworks are:

- It raises the water level on its upstream side.
- It regulates the supply of water into canals.
- It controls the entry of silt into canals.
- It forms a small pond (not reservoir) on its upstream.
- It helps in controlling the unpredictability of the river.

Site Selection for Headworks :

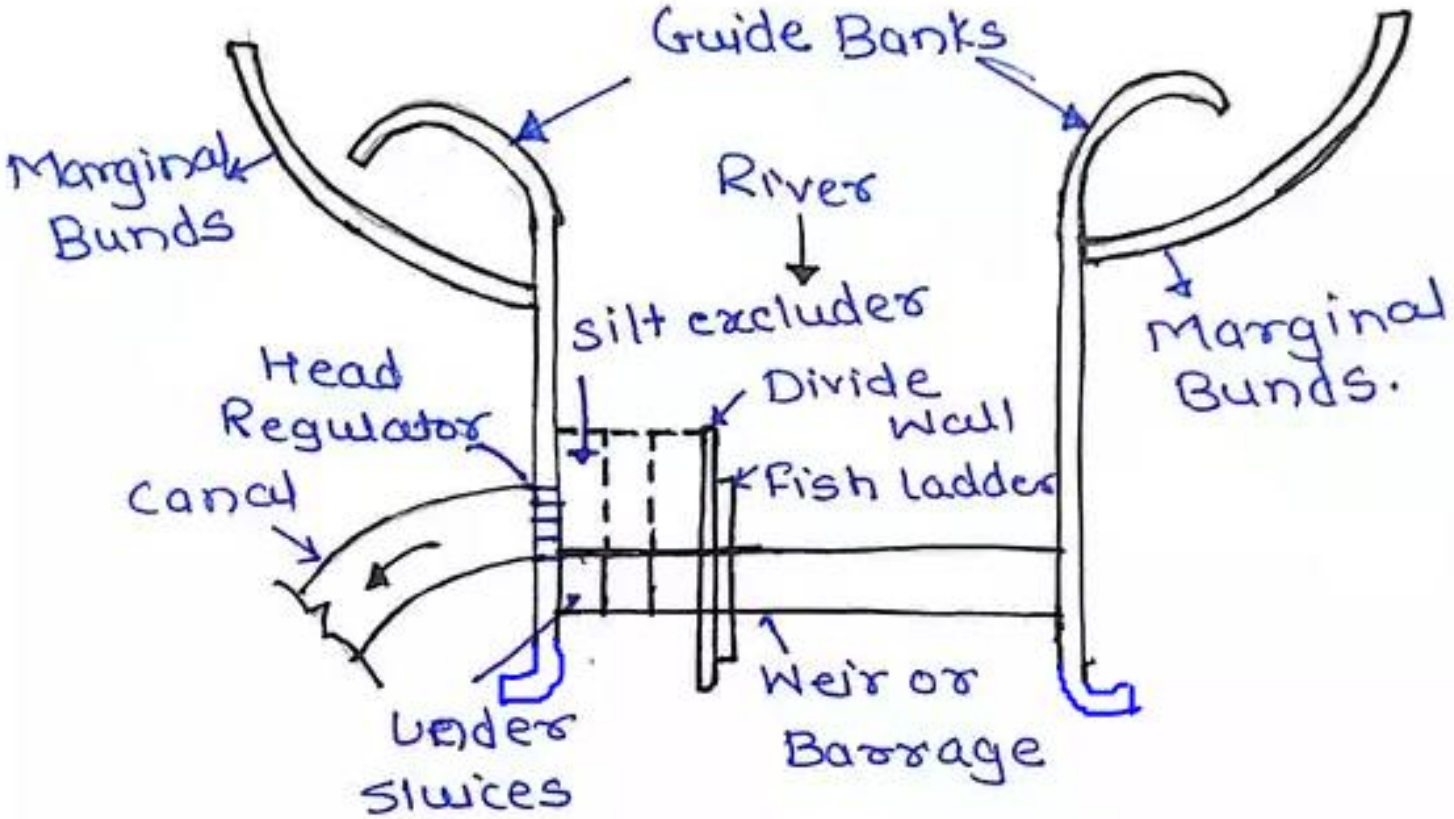
The major factors that should be considered while selecting site for headworks are:

- The river section at the site should be straight and narrow.
- The elevation of site should be higher than the area to be irrigated for gravity flow.
- The river banks should be well defined and stable.
- Good foundation should be available at the site.
- Valuable land upstream of weir or barrage should not be submerged.
- Site should be close to cropland to minimize the loss.
- Should have the large command area.
- There must be connection with transportation.

Components of a Diversion Headworks

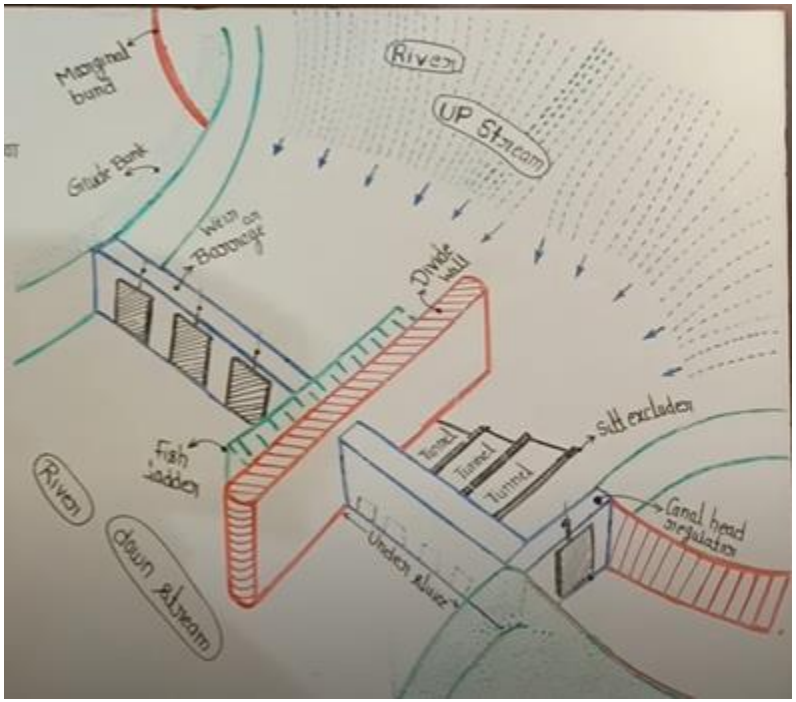
- ❑ Weir or Barrage
- ❑ Undersluices
- ❑ Divide wall
- ❑ Fish Ladder
- ❑ Canal head regulator
- ❑ Silt excluder/Silt prevention devices
- ❑ River training works

Component Parts of Diversion Headworks:

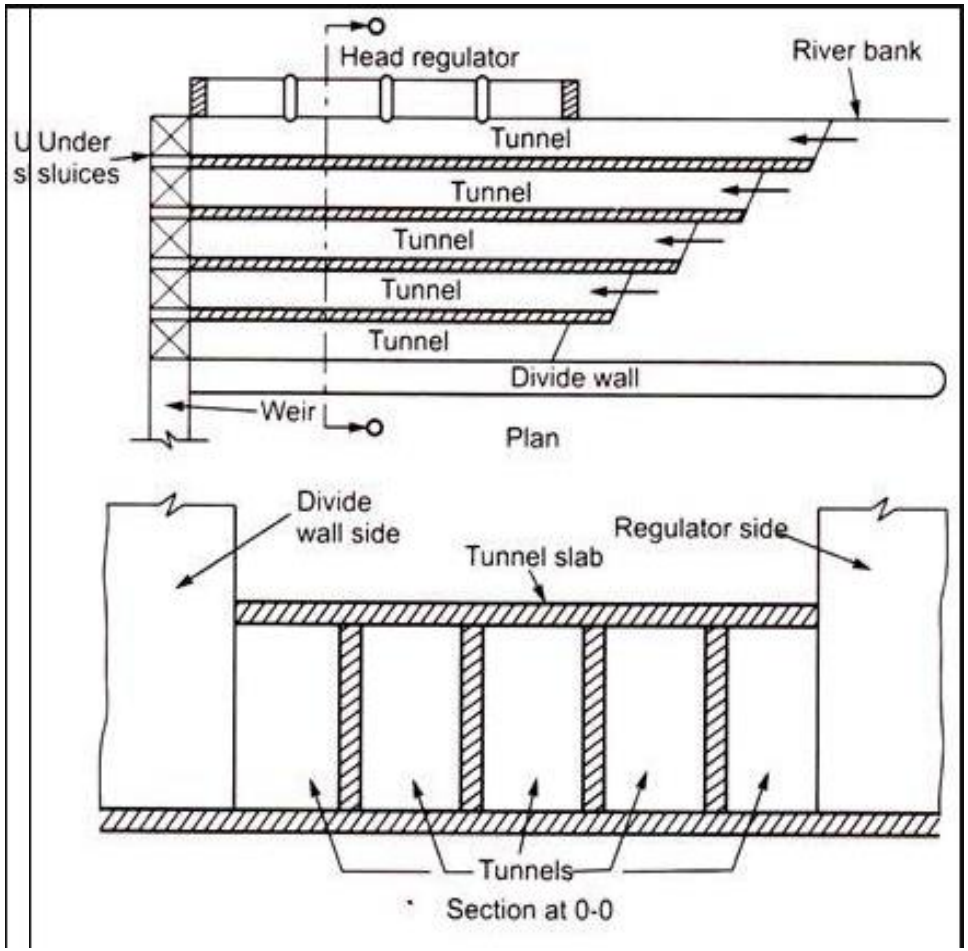


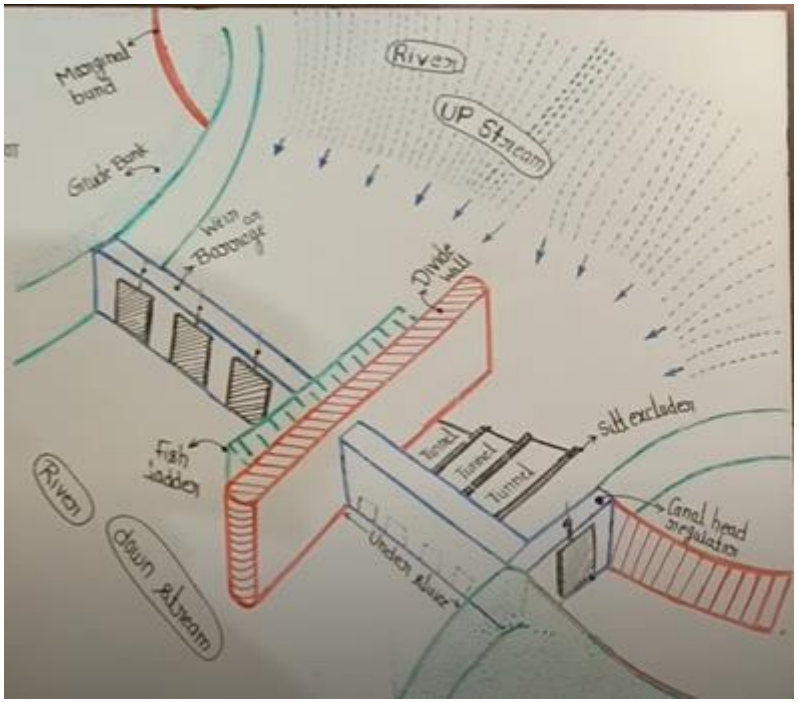
* Layout of Diversion Headworks *

❖ Components of Diversion Headwork



❖ Undersluices and silt excluder





Under Sluices

Weir

- Normally the water level of any perennial river is such that it cannot be diverted to the irrigation canal.
- The bed level of the canal may be higher than the existing water level of the river.
- In such cases weir is constructed across the river to raise the water level.
- Surplus water pass over the crest of weir.
- Adjustable shutters are provided on the crest to raise the water level to some required height.

Weir

Barrage

- When the water level on the up stream side of the weir is required to be raised to different levels at different time, barrage is constructed.

- Barrage is an arrangement of adjustable gates or shutters at different tires over the weir.

PAMBA
IRRIGATION
PROJECT
MANIYAR

Comparisons between weir and barrage

Weir

- Low control on flow
- No provision for transport communication across the river
- Chances of silting on the upstream is more
- Low cost
- Shorter construction period

Barrage

- Relatively high control on flow and water levels by operation of gates
- Road or bridge can be conveniently and economically combined with barrage
- Silting may be controlled by judicial operation of gates
- High cost
- Longer construction period

Vertical Drop Weir

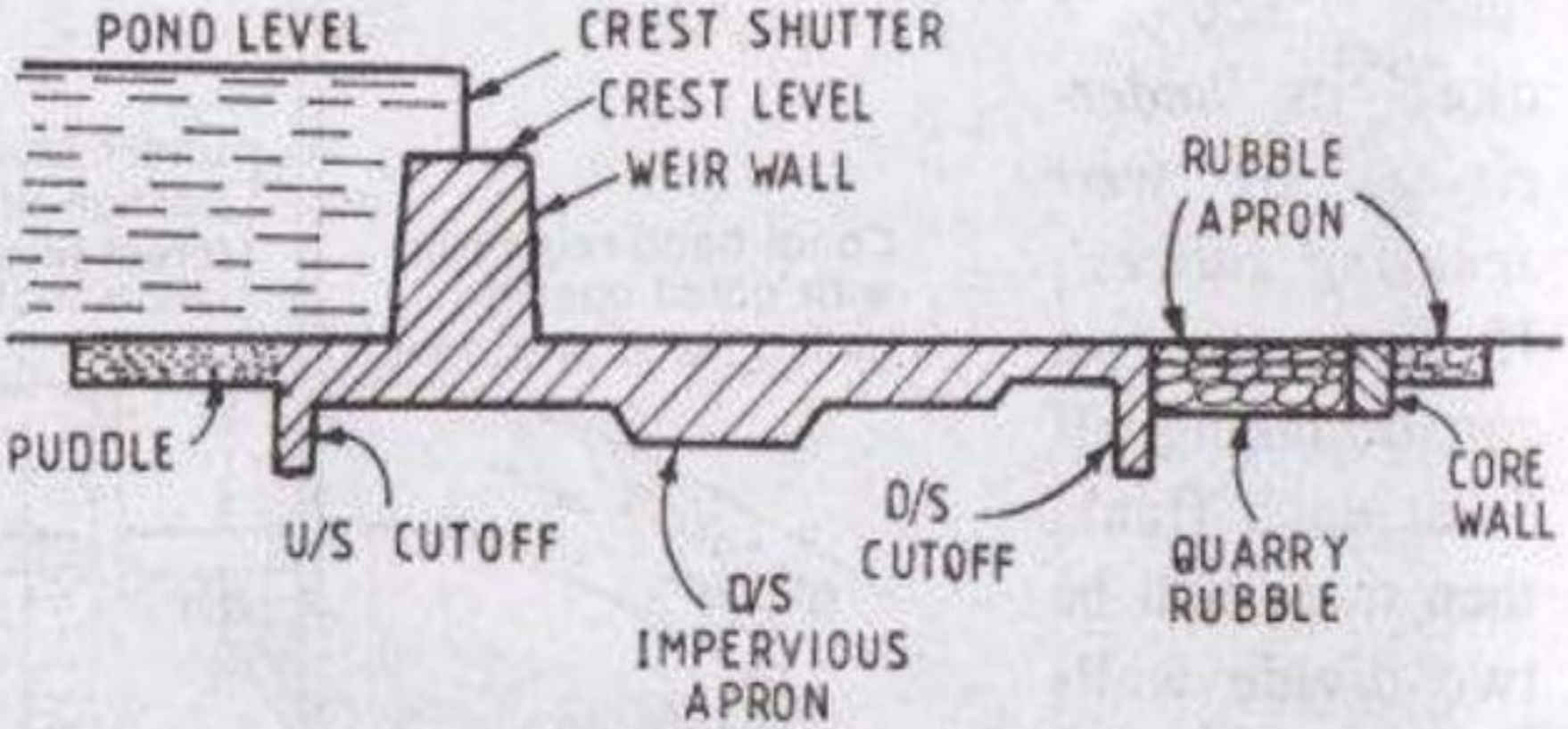


Fig. Masonry weir

Sloping Weir of Concrete

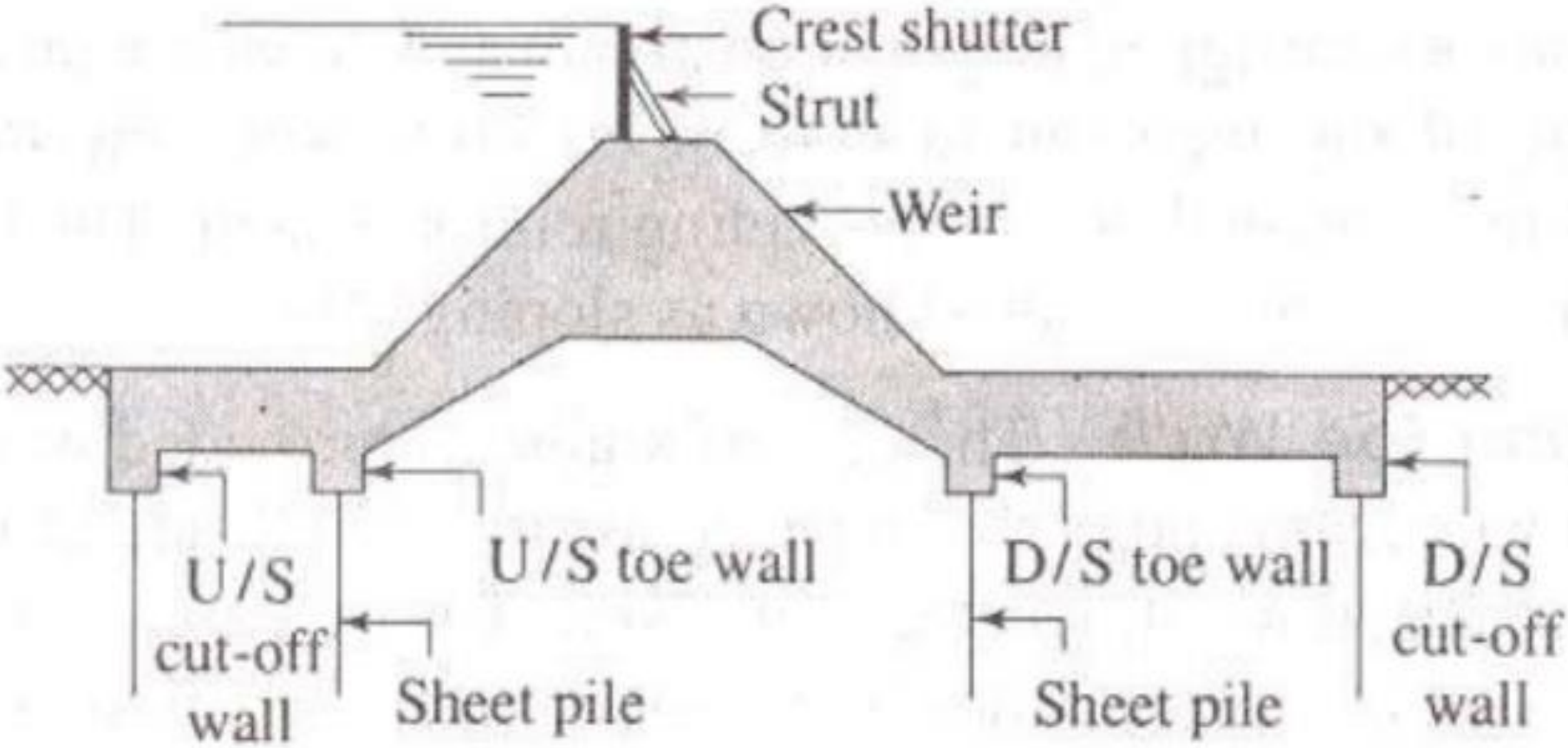


Fig: Concrete Weir

Dry Stone Slopping Weir

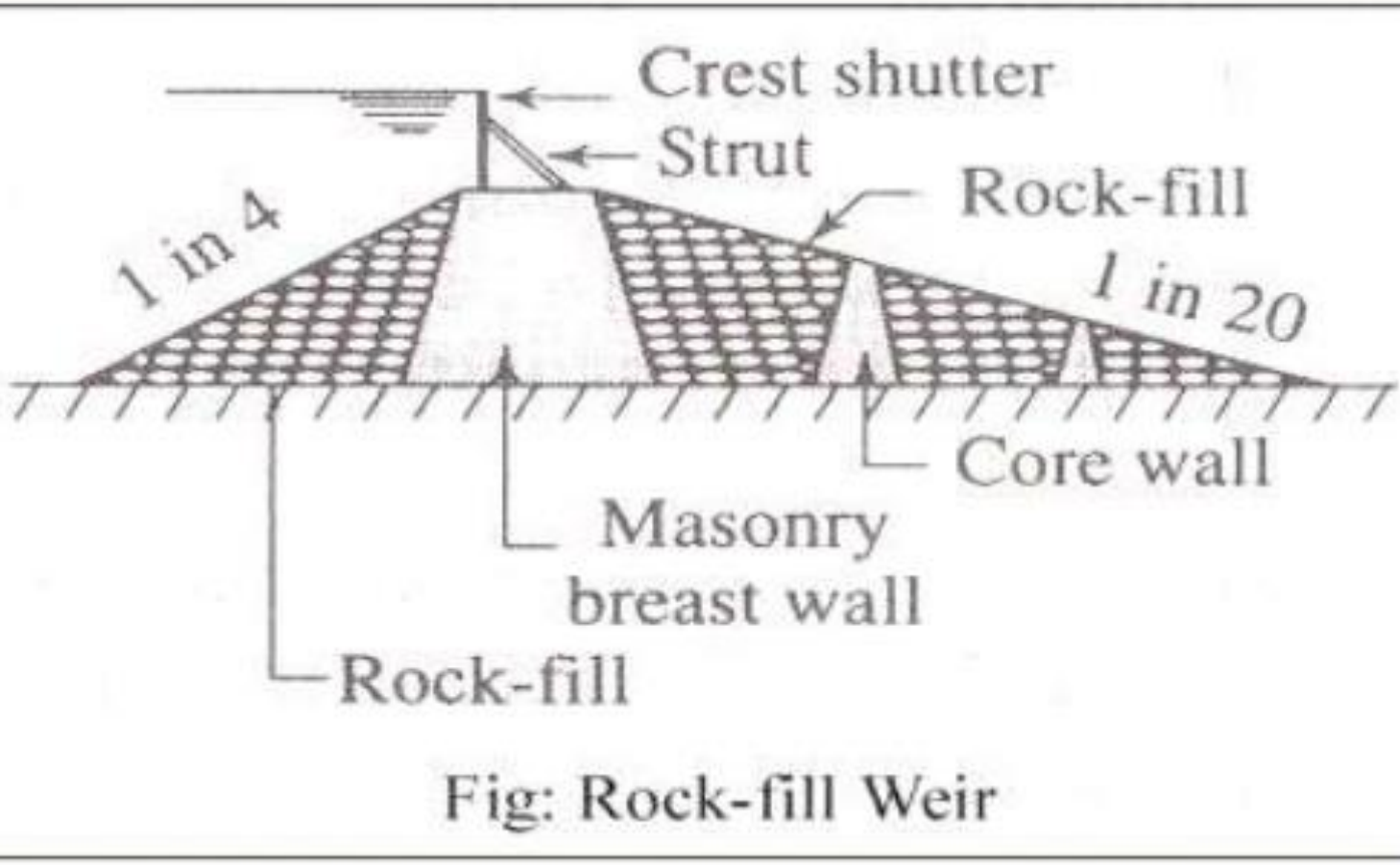


Fig: Rock-fill Weir

2. Undersluice

- Also known as scouring sluices.
- The under sluices are the opening provided at the base of the weir or barrage.
- The openings are provided with adjustable gates. Normally, the gates are kept closed.
- The suspended silt goes on depositing in front of the canal head regulator.
- When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounting on a boat.
- The muddy water flows towards the downstream through the scouring sluices.
- The gates are then closed. But at the period of flood, the gates are kept opened.

Functions:

i. To scour away silt deposited in front of head regulator.

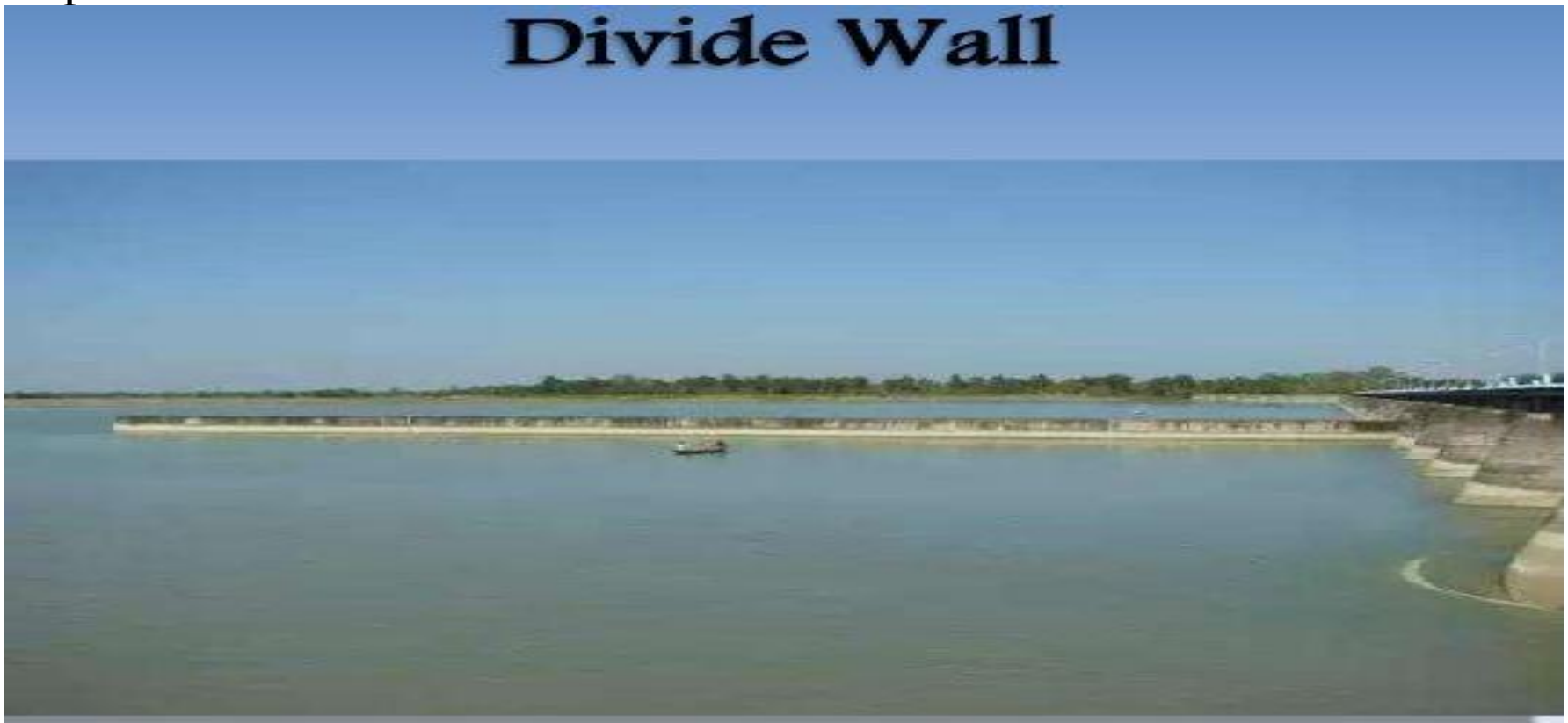
ii. To pass about 10% to 20% of design flood during rainy season.

iii. To maintain well maintained river channel near canal head regulator.

Under Sluices

3. Divide Wall

- It is the long wall constructed at right angle to the weir or barrage on u/s side.
- It may be constructed with stone masonry or cement concrete.
- On the u/s side, the wall is extended just to cover the canal head regulator and on the d/s side, it is extended up to the launching apron.

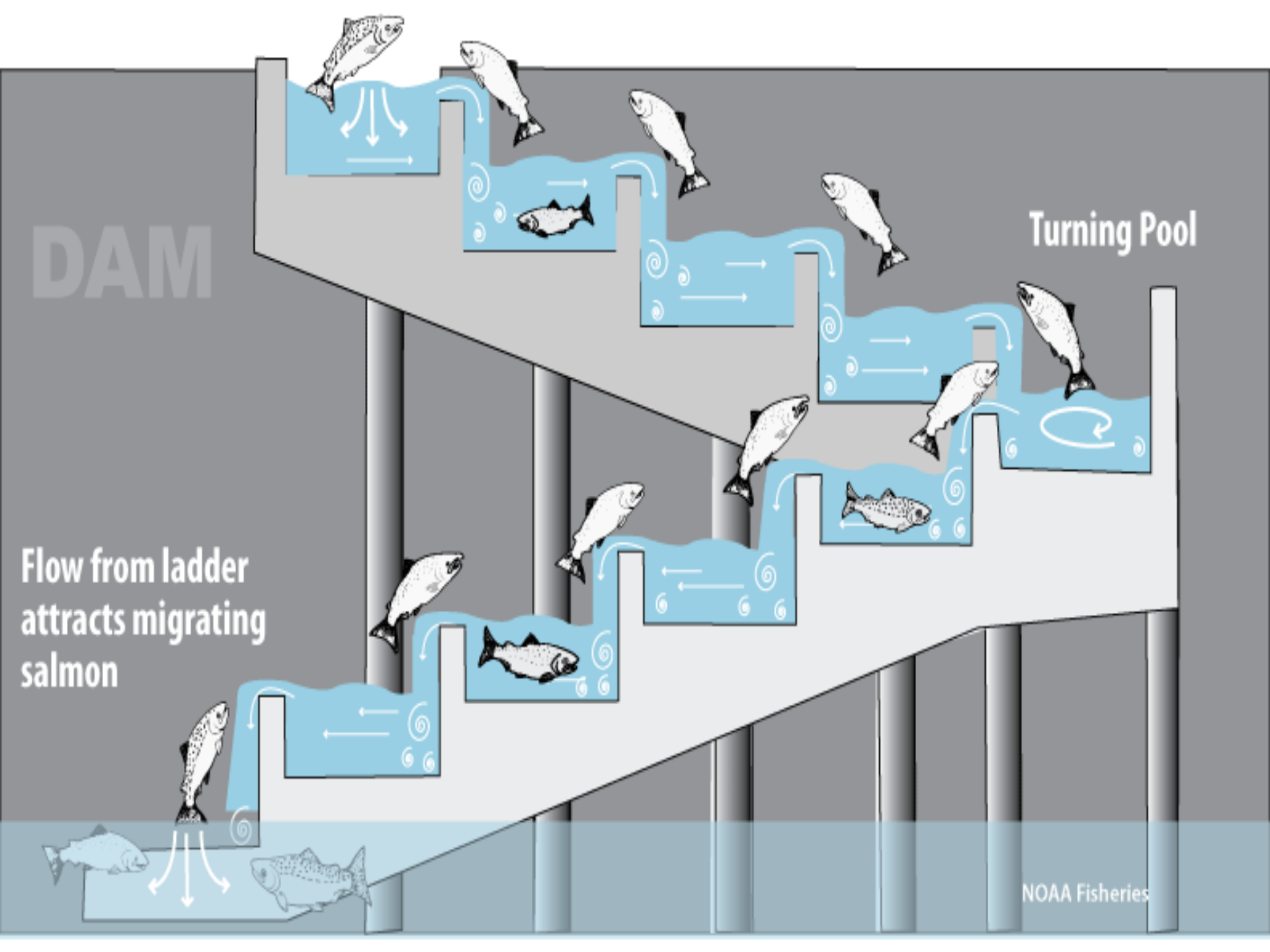


Functions of Divide Wall

- To form a still water pocket in front of the canal head so that the suspended silt can be settled which then later be cleaned through the scouring sluices from time to time.
- To separate the undersluice portion from weir portion.
- It provides straight approach in front of the canal head.
- To prevent cross current and flow parallel to the weir.
- Side walls of the fish ladder.

Fish Ladder/Fish Passage

- The fish ladder is provided just by the side of the divide wall for the free movement of fishes.
- Rivers are important sources of fishes.
- The tendency of fish is to move from upstream to downstream in winters and from downstream to upstream in monsoons.
- This movement is essential for their survival.
- Due to construction of weir or barrage, this movement gets obstructed, and is detrimental to the fishes.
- The width. Length and height of the fish ladder depend on the nature of the river and the type of the weir or barrage.


DAM
Turning Pool
Flow from ladder attracts migrating salmon
NOAA Fisheries

Fish Ladder

Activate Windows
Go to Settings to activate

Canal head regulator

- A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator.
- It consists of a number of piers which divide the total width of the canal into a number of spans which are known as bays.
- The piers consist of number tiers on which the adjustable gates are placed.

- The gates are operated form the top by suitable mechanical device.
- A platform is provided on the top of the piers for the facility of operating the gates.
- Again some piers are constructed on the down stream side of the canal head to support the roadway.

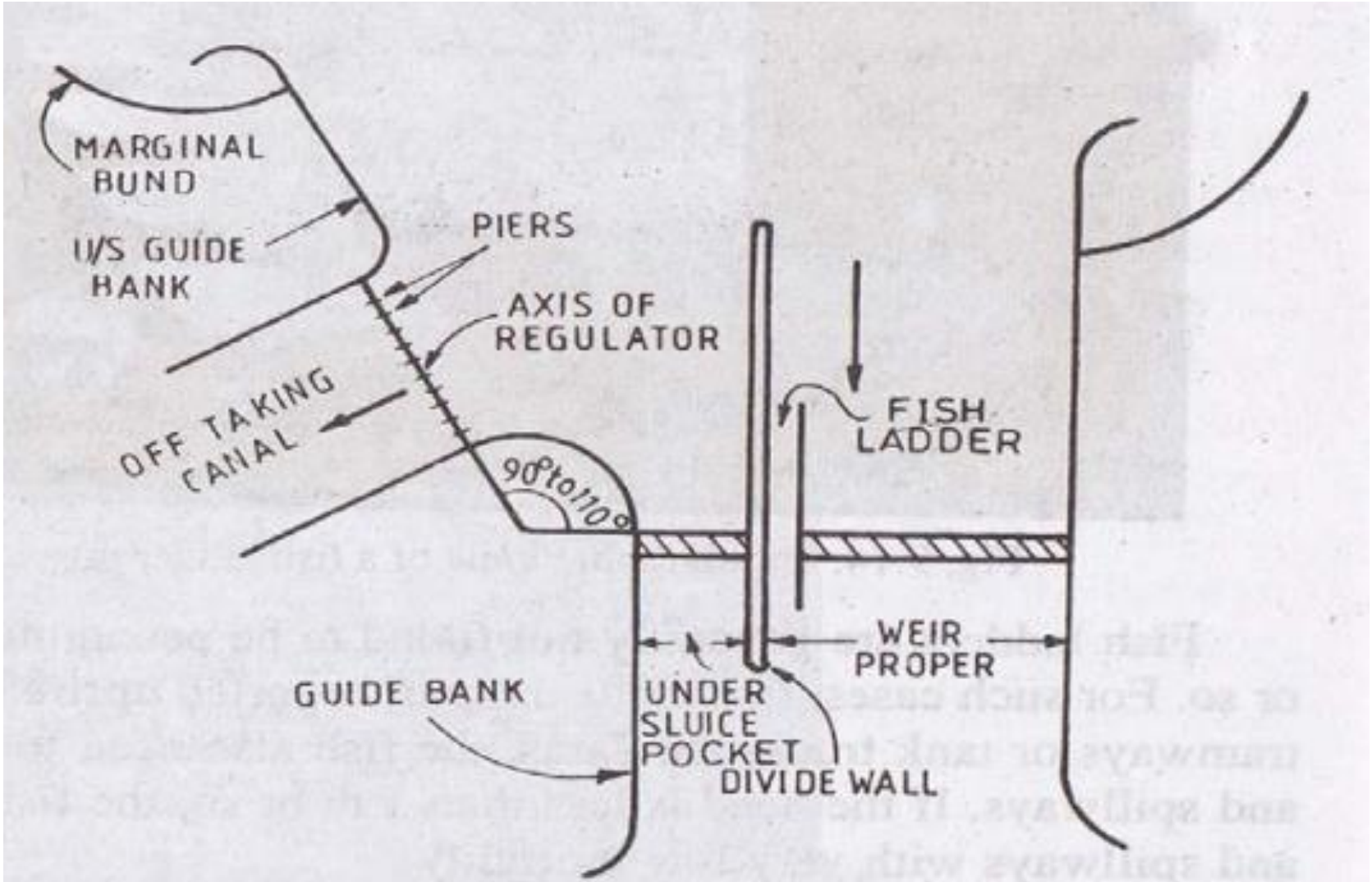
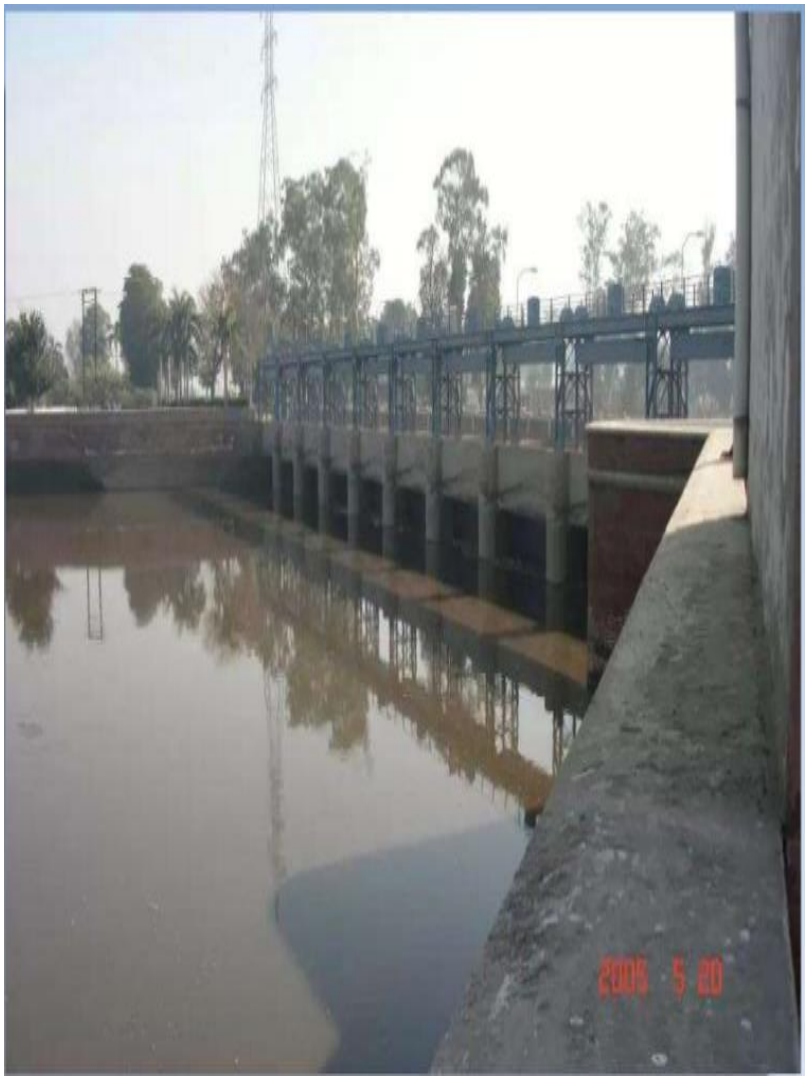
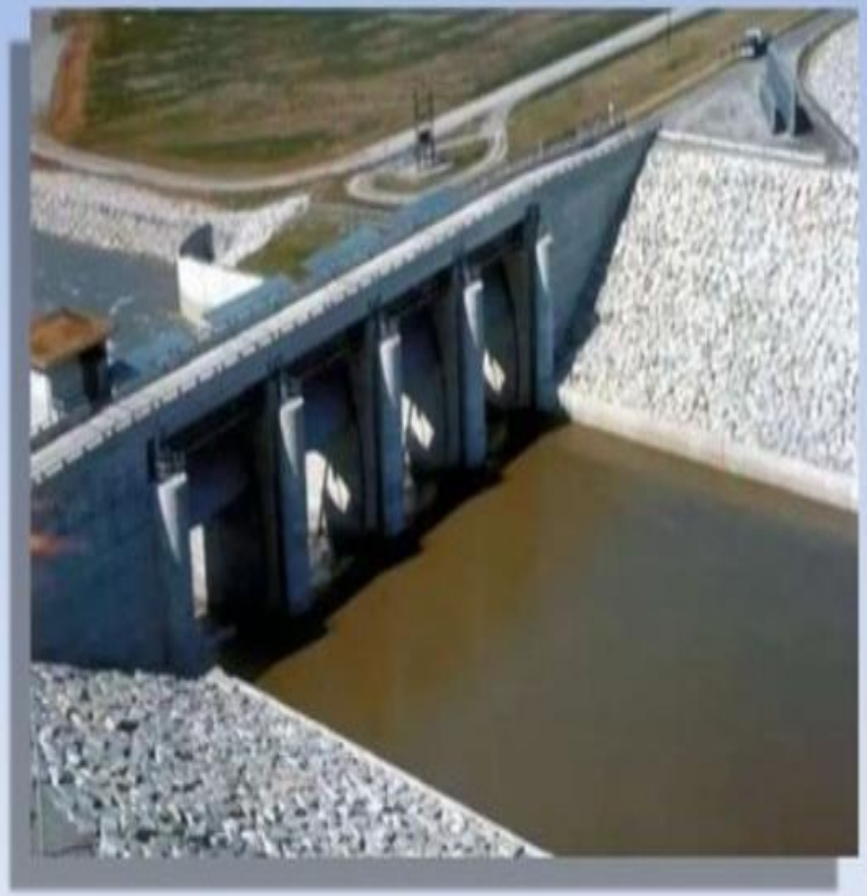


Fig: Alignment of a canal head regulator



Head Regulator



Functions of Canal Head Regulator

- It regulates the supply of water entering the canal
- It controls the entry of silt in the canal
- It prevents the river-floods from entering the canal

Silt regulation works

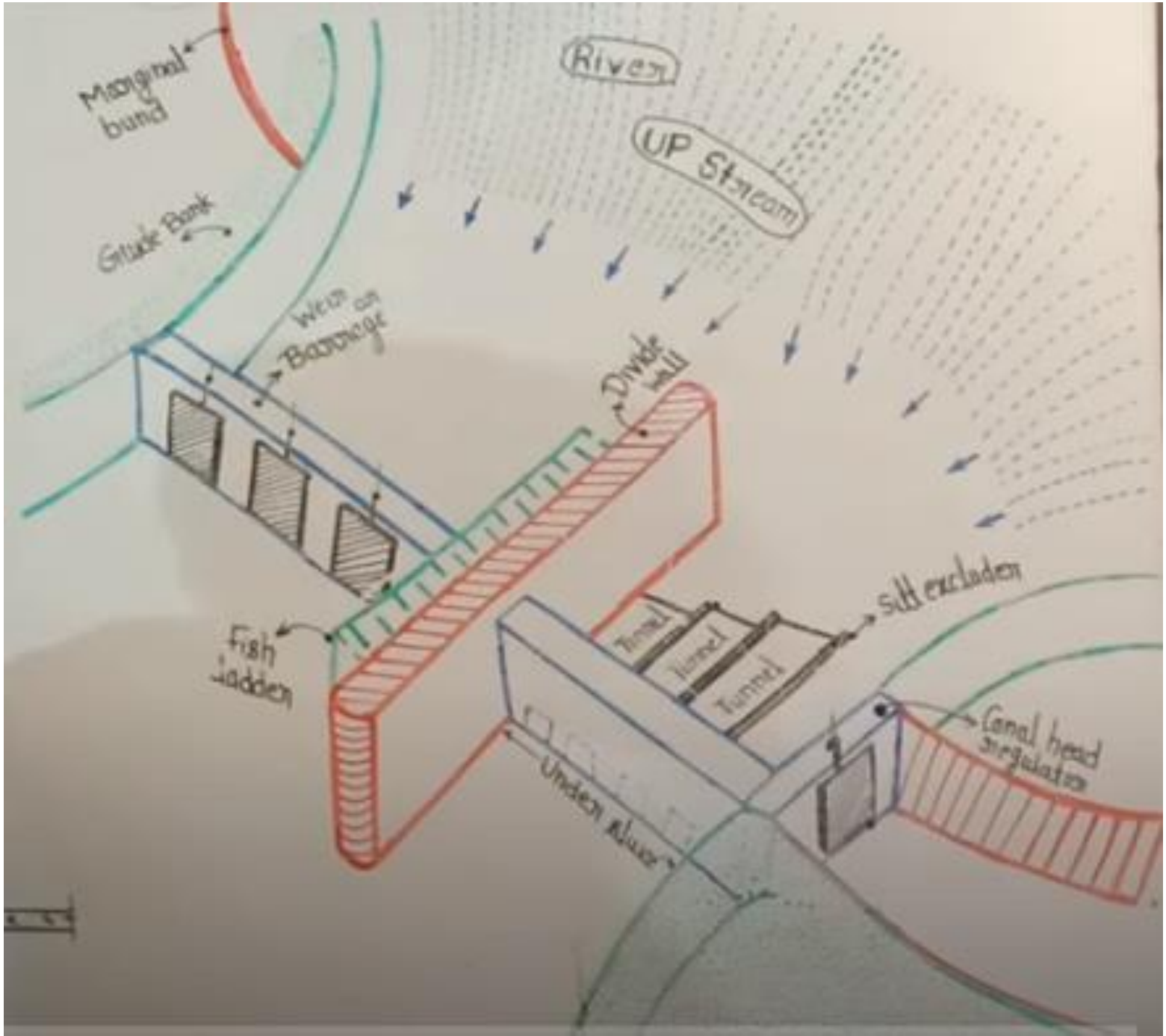
- The entry of silt into a canal, which takes off from a head works, can be reduced by constructed certain special works, called silt control works.
- These works may be classified into the following two types:

  (a) Silt Excluders

  (b) Silt Ejectors

Silt Excluders

- Silt excluders are those works which are constructed on the bed of the river, upstream of the head regulator.
- The clearer water enters the head regulator and silted water enters the silt excluder.
- In this type of works, the silt is, therefore,, removed from the water before in enters the canal.


Marginal bund
River
UP Stream
Guide Bank
Weir or Barrage
Divide wall
Fish ladder
Silt excluder
Tunnel
Tunnel
Tunnel
Canal head regulator
Under sluice

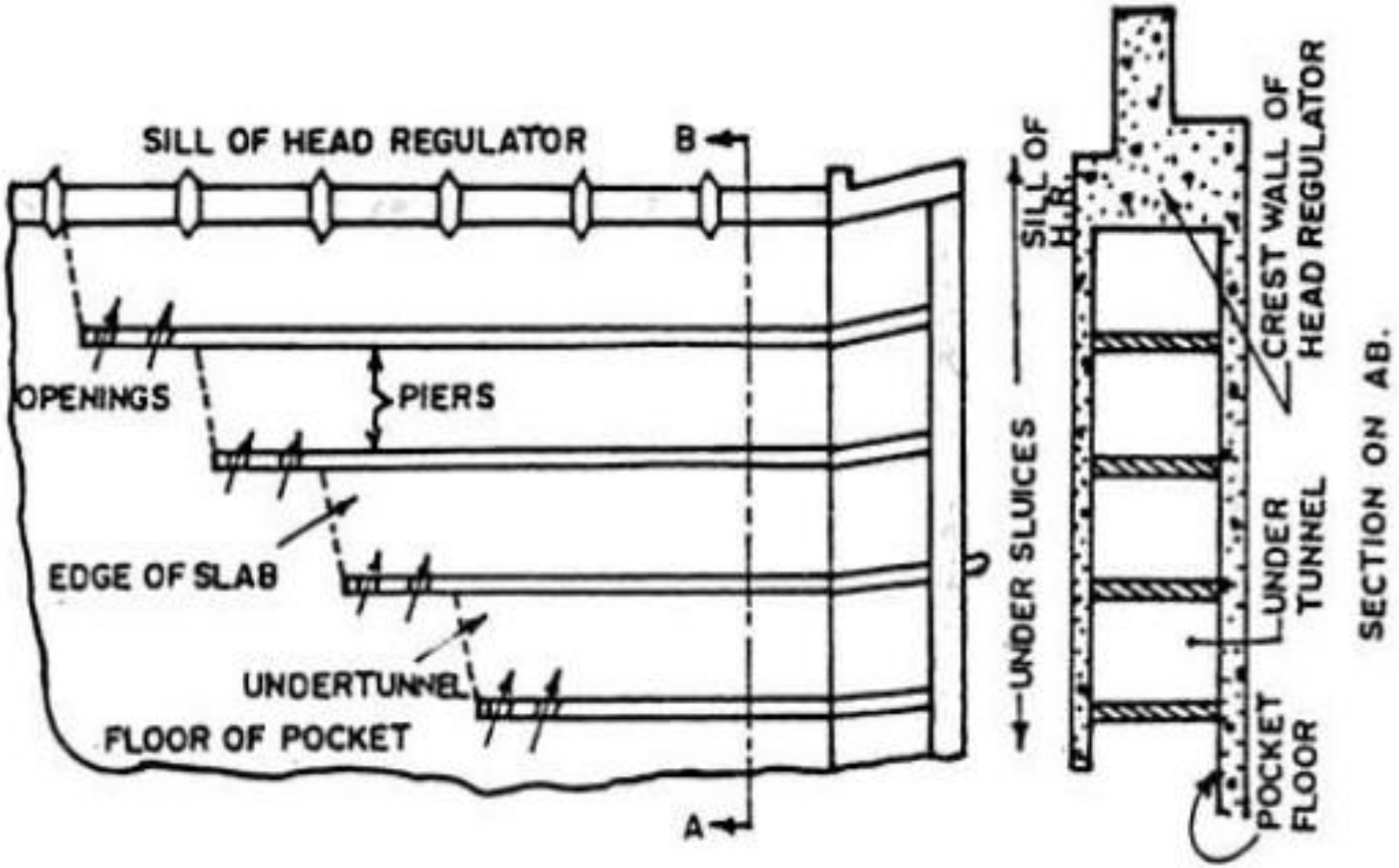


FIG. 12.28. SILT EXCLUDER.

Fig. 12.28 shows a silt excluder —a type used at Khanki weir.

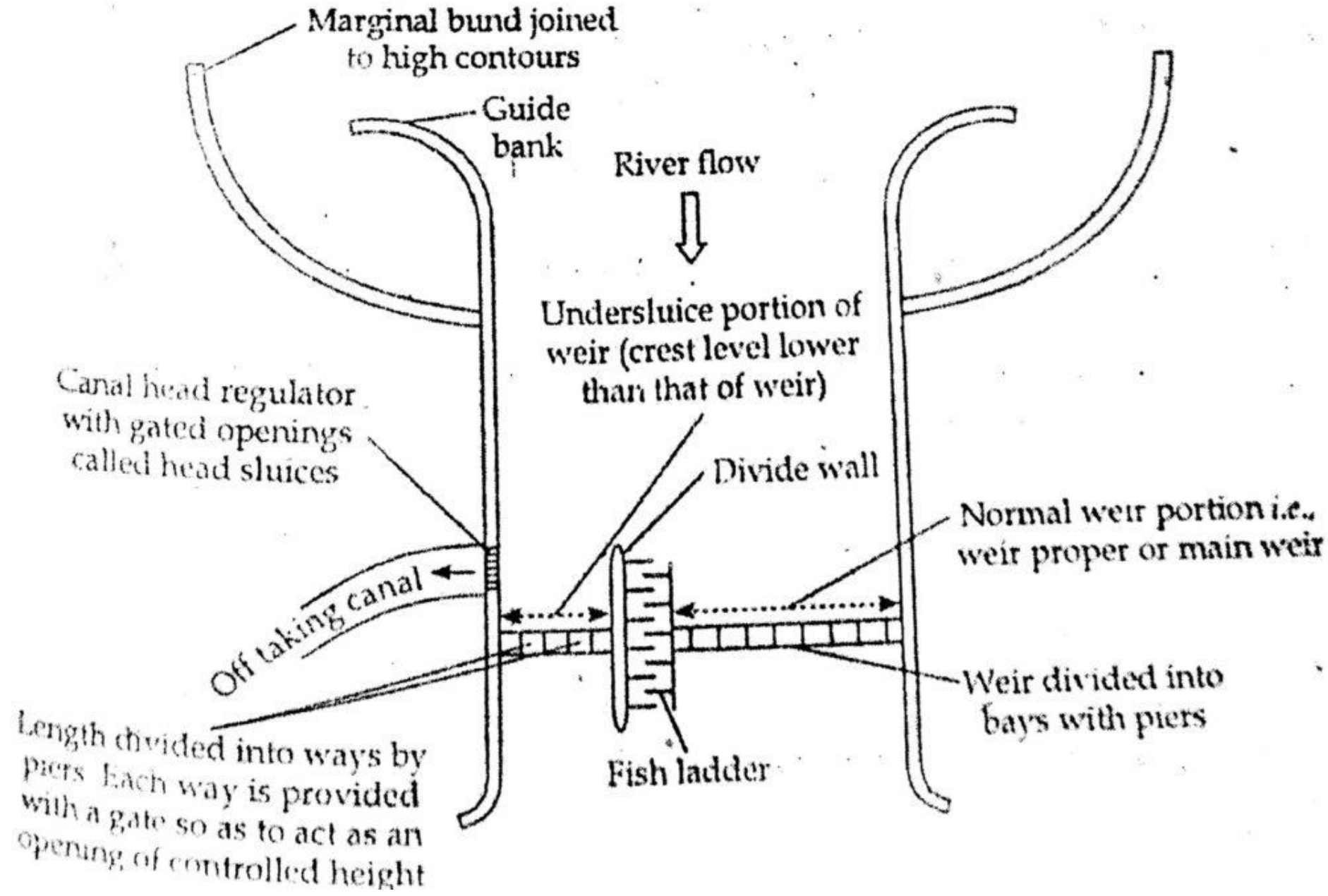
Silt Ejectors

- Silt ejectors, also called silt extractors, are those devices which extract the silt from the canal water after the silted water has travelled a certain distance in the off-take canal.

- These works are, therefore, constructed on the bed of the canal, and little distance downstream from the head regulator.

River training works

- River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from outflanking the works due to a change in its course.
- The river training works required on a canal headwork are:
    - (a) Guide banks
    - (b) Marginal bunds
    - (c) Spurs or groynes


Marginal bund joined to high contours
Guide bank
River flow
Undersluice portion of weir (crest level lower than that of weir)
Canal head regulator with gated openings called head sluices
Divide wall
Normal weir portion i.e., weir proper or main weir
Off taking canal
Weir divided into bays with piers
Length divided into ways by piers Each way is provided with a gate so as to act as an opening of controlled height
Fish ladder

Causes of Failures of Weirs on Permeable Foundations

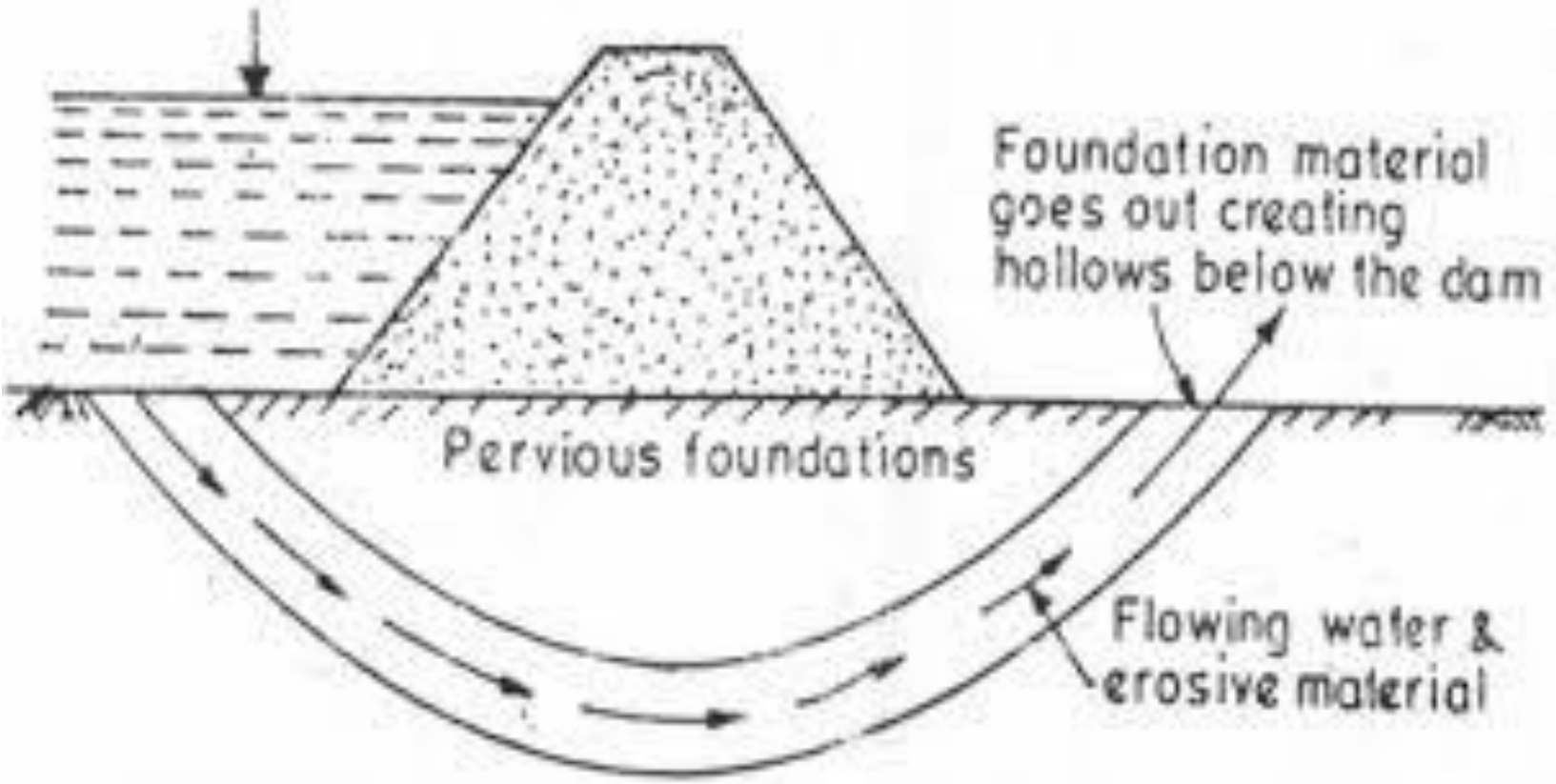
Causes of Failure:

1. Due to Seepage or Sub-surface Flow
   a) Piping or Undermining
   b) Rupture of Floor by Uplift Pressure
2. Due to Surface Flow
   a) By Suction due to Hydraulic Jump
   b) By Scour on the u/s and d/s of the weir

Activate Windows

(a) Failure by Piping or undermining

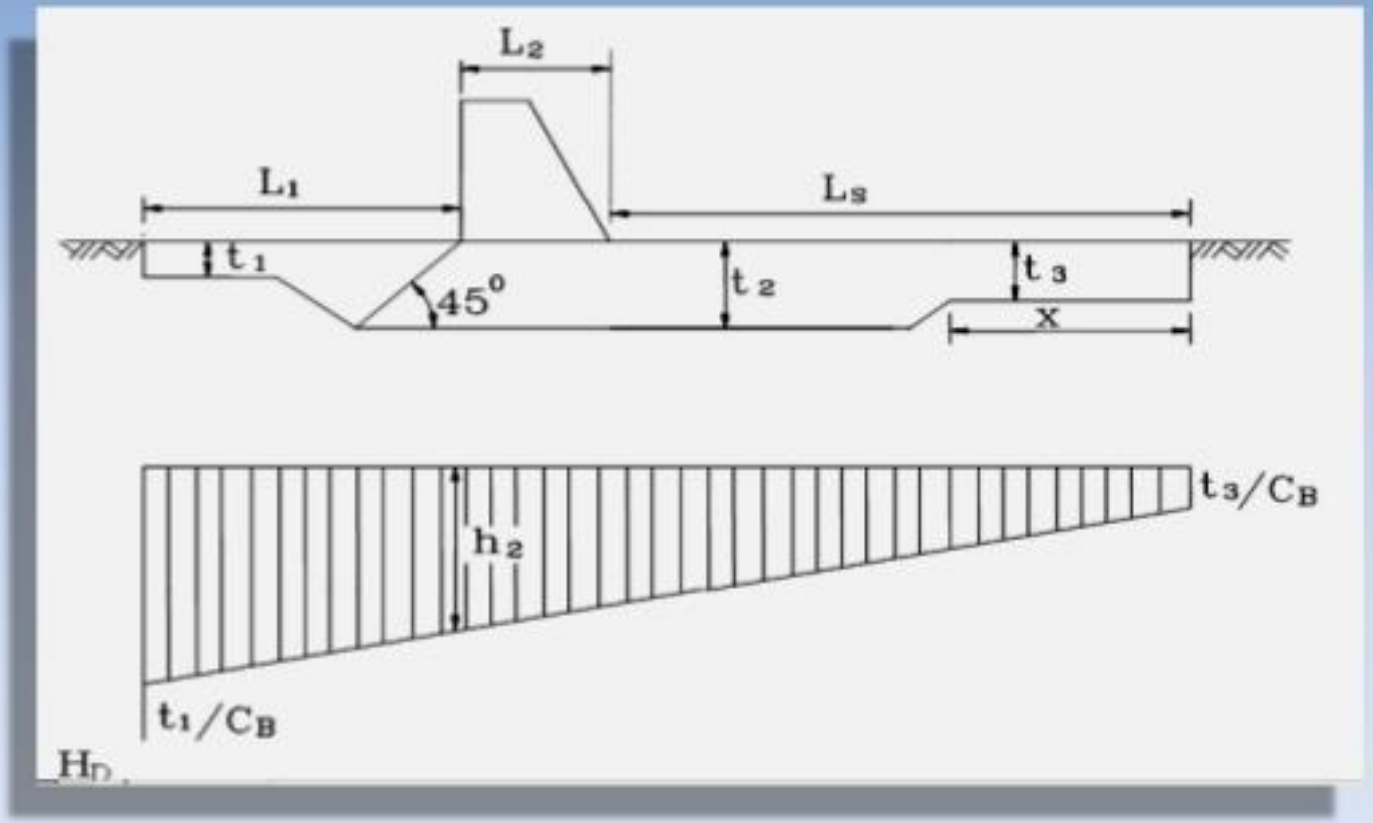
- water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor.
- The force of percolating water removes the soil particles by scouring at the point of emergence.
- As the process of removal of soil particles goes on continuously, a depression is formed which extends backwards towards the upstream through the bottom of the foundation.
- A hollow pipe like formation thus develops under the foundation due to which the weir or barrage may fail by subsiding.
- This phenomenon is known as failure by piping or undermining.


Foundation material
goes out creating
hollows below the dam
Pervious foundations
Flowing water &
erosive material

(b) Failure by Direct uplift

- The percolating water exerts an upward pressure on the foundation of the weir or barrage.
- If this uplift pressure is not counterbalanced by the self weight of the structure, it may fail by rapture.

By Uplift Pressure

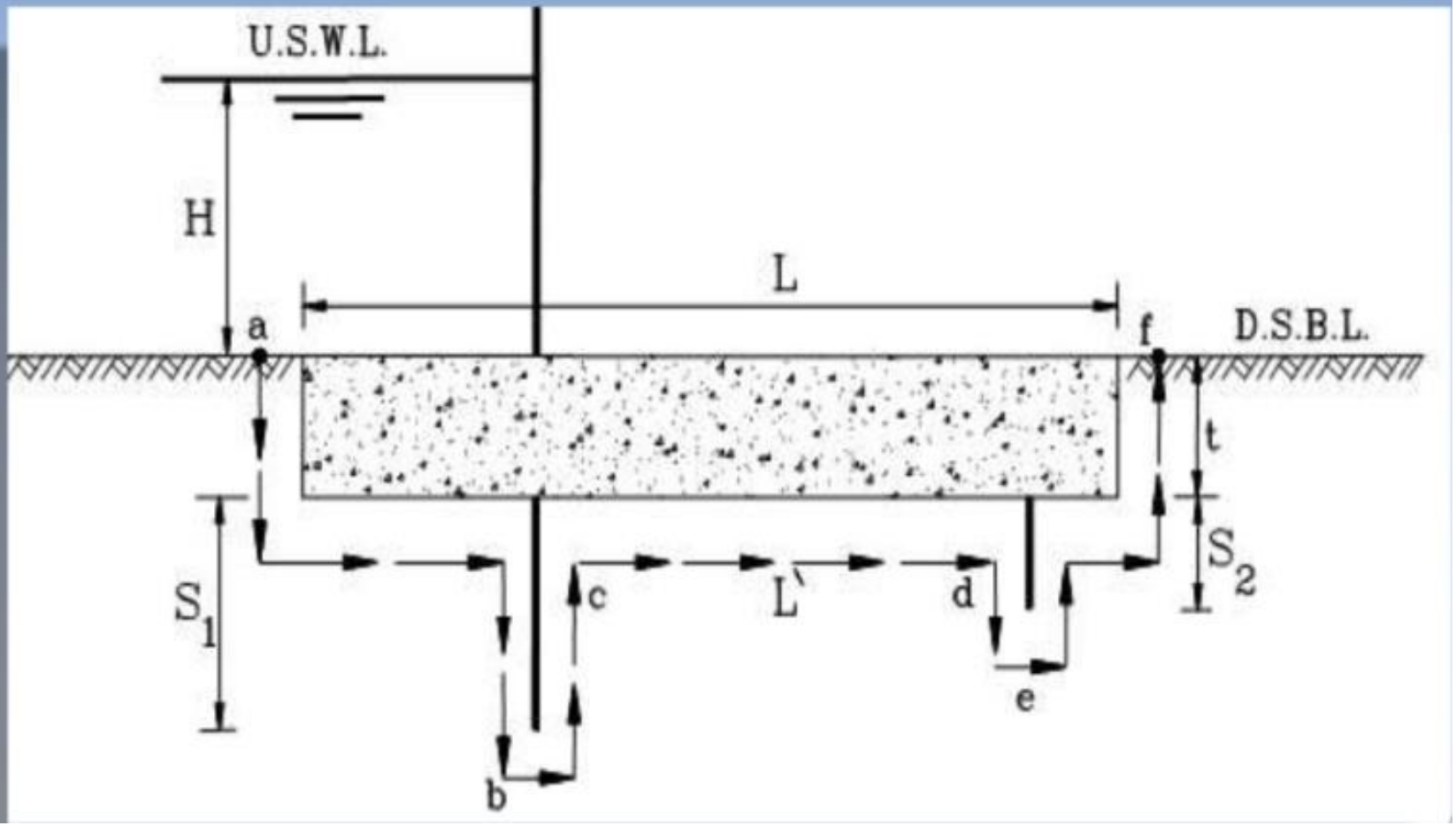


Remedial measure of this type of failures:

- Path of percolation or creep length of seepage water should be increased by providing sheet piles at upstream, downstream or at intermediate point to reduce the hydraulic gradient.

- Floor thickness should be increased to increase its self weight to balance the uplift force.

Precautions Against Failure



SEEPAGE THEORIES

Generally there are three seepage theories

1. **Bligh's creep theory**
2. **Lane weighted creep theory**
3. **Khosla theory**

❖ Bligh's creep theory

1. According to *Bligh's Theory*, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. *The length of the path thus traversed by water is called the length of the creep.*
2. Further, it is assumed in this theory, that the loss of head is proportional to the length of the creep. If H_L is the total head loss between the upstream and the downstream, and L is the length of creep, *then the loss of head per unit of creep length (i.e. H_L/L) is called the hydraulic gradient.*
3. Further, Bligh makes no distinction between horizontal and vertical creep.

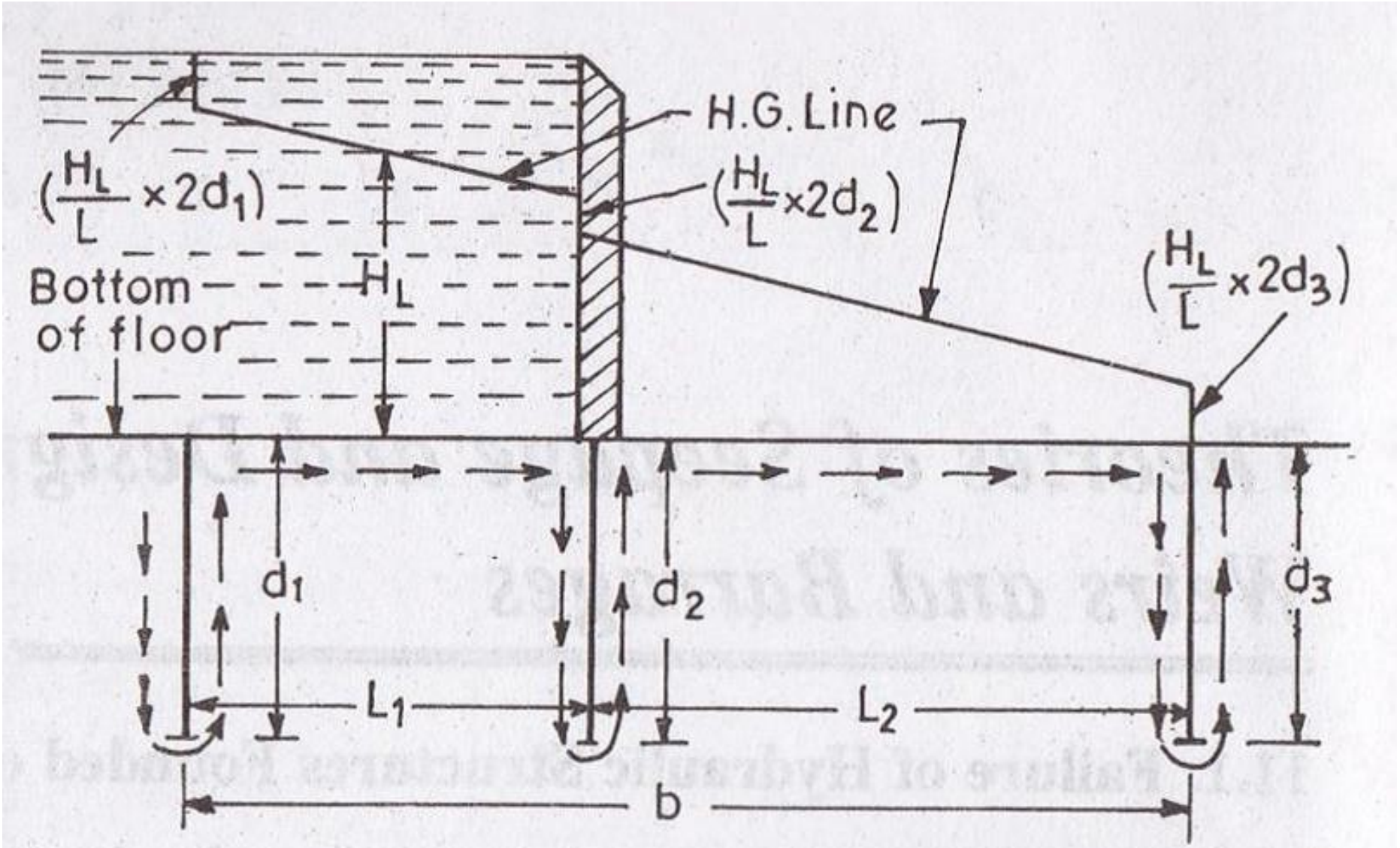
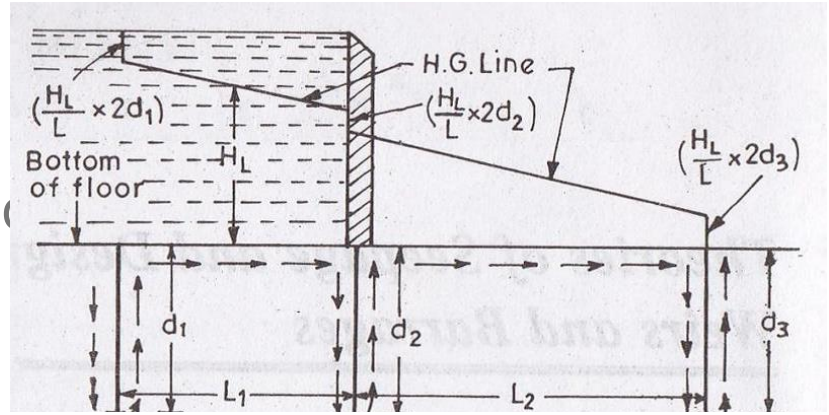


Fig-1: Bligh's Creep

- Consider a section a shown in Fig above. Let H_L be the difference of water levels between upstream and downstream ends. Water will seep along the bottom contour as shown by arrows. It starts percolating at A and emerges at B. The total length of creep is given by



L = d1 + d1 + L1 + d2 + d2 + L2 + d3 +

= (L1+ L2) + 2(d1 + d2 + d3

= b + 2(d1 + d2 + d3)

Head loss per unit length or hydraulic gradient $= \left[\frac{H_L}{b+2(d_1+d_2+d_3)}\right] = \frac{H_L}{L}$

Head losses equal to $\left(\frac{H_L}{L}\times 2d_1\right), \left(\frac{H_L}{L}\times 2d_2\right), \left(\frac{H_L}{L}\times 2d_3\right)$; will occur respectively, in the planes of three vertical cut offs. The hydraulic gradient line (H.G. Line) can then be drawn as shown in figure above.

• (a) Safety against piping or undermining;

•According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by

$$L = C.H_L,$$

where C is the Bligh's Coefficient for the soil.

Different values of C for different types of soils are tabulated in Table –1 below:

SL No.	Type of Soil	Value of C	Safe Hydraulic gradient should be less than
1	Fine micaceous sand	15	1/15
2	Coarse grained sand	12	1/12
3	Sand mixed with boulder and gravel, and for loam soil	5 to 9	1/5 to 1/9
4	Light sand and mud	8	1/8

•

Note: The hydraulic gradient i.e. H_L/L is then equal to 1/C. Hence, it may be stated that the hydraulic gradient must be kept under a safe limit in order to ensure safety against piping.

(b) Safety against uplift pressure

The ordinates of the H.G line above the bottom of the floor represent the residual uplift water head at each point.

- Say for example, if at any point, the ordinate of H.G line above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If h′ meters is this ordinate, then water pressure equal to h′ meters will act at this point, and has to be counterbalanced by the weight of the floor of thickness say t.

Uplift pressure = $\gamma_w \times h'$ [where γ_w is the unit weight of water]

- Downward pressure = $(\gamma_w \times G).t$ [Where G is the specific gravity of the floor material]

For equilibrium,

- $\gamma_w \times h' = \gamma_w \times G.\ t$
- $h' = G \times t$
- Subtracting t on both sides, we get

$(h' - t) = (G \times t - t) = t\,(G - 1)$

$$\Rightarrow t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)$$

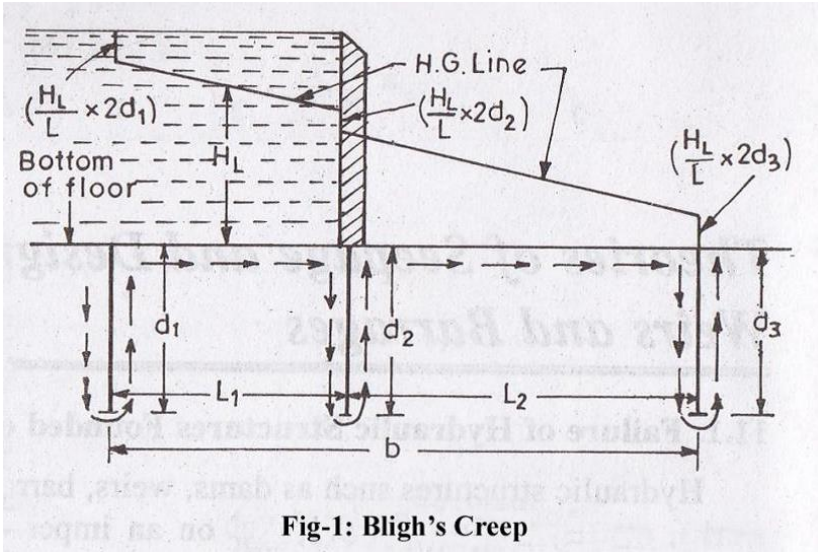


Fig-1: Bligh's Creep

- Where, h′ – t = h = Ordinate of the H.G line above the top of the floor
- G – 1 = Submerged specific gravity of the floor material

❖ Lane's Weighted Creep Theory

1. Bligh, in his theory, had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps.
2. However, Lane, on the basis of his analysis carried out on about 200 dams all over the world, stipulated that horizontal creep is less effective in reducing uplift (or in causing loss of head) than vertical creep.
3. He, therefore, suggested a weightage factor of 1/3 for the horizontal creep, as against 1.0 for the vertical creep.

the total Lane's creep length (Ll) is given by

$L_l = (d1 + d1) + (1/3)\,L1 + (d2 + d2) + (1/3)\,L2 + (d3 + d3)$

$= (1/3)\,(L1 + L2) + 2(d1 + d2 + d3)$

$= (1/3)\,b + 2(d1 + d2 + d3)$

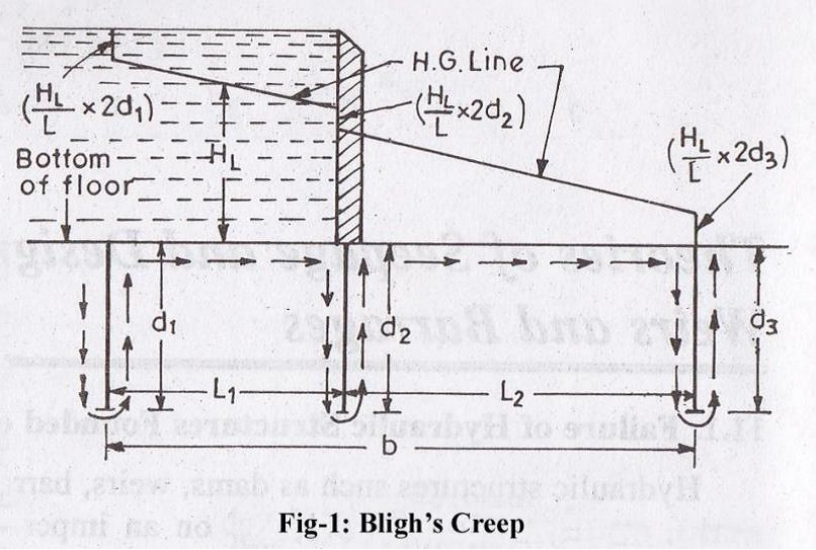


Fig-1: Bligh's Creep

To ensure safety against piping, according to this theory, the creep length L_l must not be less than $C_1 H_L$, where H_L is the head causing flow, and C_1 is Lane's creep coefficient given in table –2

SL No.	Type of Soil	Value of Lane's Coefficient C_1	Safe Lane's Hydraulic gradient should be less than
1	Very fine sand or silt	8.5	1/8.5
2	Fine sand	7.0	1/7
3	Coarse sand	5.0	1/5
4	Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
5	Boulders, gravels and sand	2.5 to 3.0	1/2.5 to 1/3
6	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

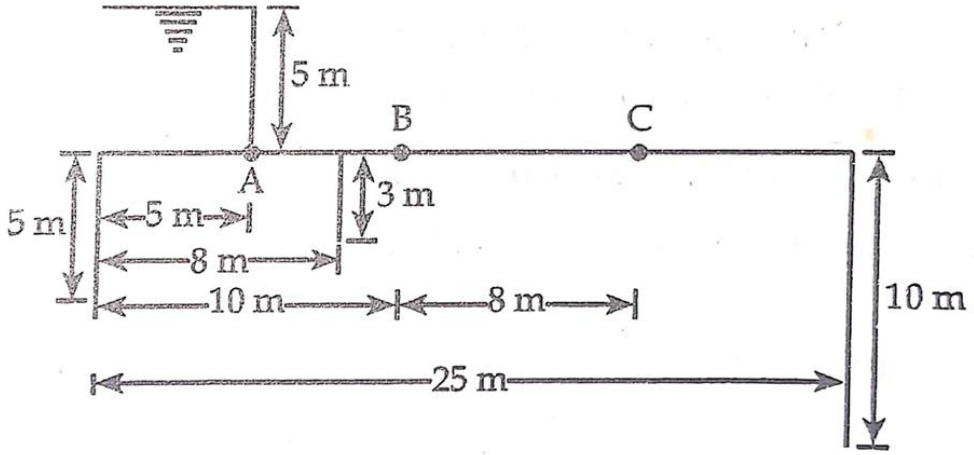
s of

The figure below shows the section of hydraulic structure on permeable foundation. Calculate the average hydraulic gradient using,

i) Bligh's creep theory

ii) Lane's weighted creep theory

Also find uplift pressure and floor thickness required at point A, B and C from both the theories.



Solution:

i) **According to Bligh's theory**

Total creep length (L) $= (5 \times 2) + 8 + (3 \times 2) + (25 - 8) + (10 \times 2) = 61\,\text{m}$

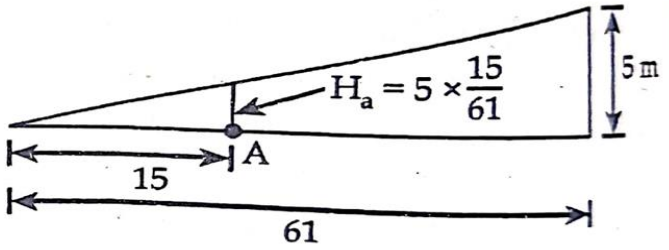
Total head available $= 5\,\text{m}$

$\therefore$ Hydraulic gradient $= \frac{5}{69} = \frac{1}{12.2}$

a) **Uplift pressure and floor thickness point A,**

Creep length up to A $= (5 \times 2) + 5$

$= 15$



Residual head at point A,

$$h_1 = 5 - 5 \times \frac{15}{61} = 5\left(1 - \frac{15}{61}\right) = 3.77\,\text{m}$$

$\therefore$ Uplift pressure @ A $= 9.81\,\text{kN/m}^3 \times 3.77\,\text{m}$

$= 36.984\,\text{kN/m}^2$

Thickness of floor at point A,

$$t_1 = \frac{h_1}{G_C - 1}$$ [From equation (5.2)]

Increasing by 33%;

$t_1 = 1.33 \times 3.04 = 4.04\,\text{m}$

b) **Uplift pressure and floor thickness at B,**

Creep length up to B $= (5 \times 2) + 8 + (3 \times 2) + (10 - 8) = 26\,\text{m}$

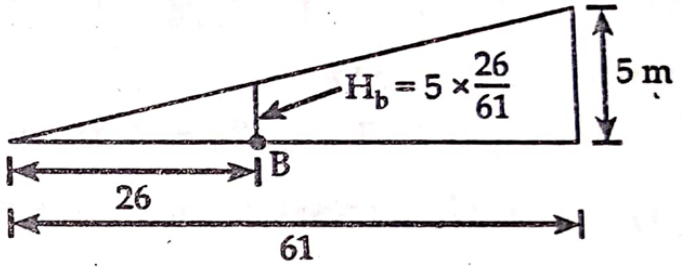
Residual heat at point B,

$$h_2 = 5 - 5 \times \frac{26}{61} = 5\left(1 - \frac{26}{61}\right) = 2.87\,\text{m}$$

$\therefore$ Uplift pressure @ B = $9.81\ kN/m^3 \times 2.87\ m = 28.155\ kN/m^2$

Thickness of floor at point B,

$$t_2 = \frac{h_2}{G_C - 1} = \frac{2.87}{2.24 - 1} = 2.315\ m$$



Increasing by 33%;

$$t_2 = 1.33 \times 2.315 = 3.079\ m$$

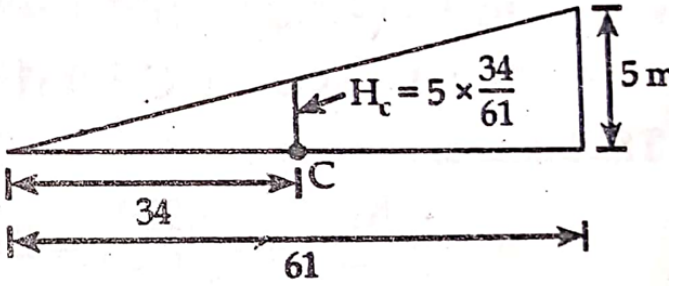
c) **Uplift pressure and floor thickness at C,**

Creep length up to C = $(2 \times 5) + 18 + (3 \times 2) = 34\ m$

Residual head at C,

$$h_3 = 5 - 5 \times \frac{34}{61}$$

$$= 5\left(1 - \frac{34}{61}\right) = 2.213\ m$$



$\therefore$ Uplift pressure @ C $= 9.81 \times 2.213$

$= 21.710\ kN/m^2$

Thickness of floor at point C,

$$t_3 = \frac{h_3}{G_C - 1} = \frac{2.213}{2.24 - 1} = 1.785\ m$$

Increasing by 33%;

$$t_3 = 1.33 \times 1.785 = 2.374$$

ii) **According to Lane's theory,**

Total creep length, $L = (5 \times 2) + (3 \times 2) + (10 \times 8) + \frac{1}{3} \times 25 = 44.333$ m

$\therefore$ Hydraulic gradient $= \frac{5}{44.333} = \frac{1}{8.867}$

a) **Uplift pressure and floor at A,**

Creep length up to $A = 5 \times 2 + \frac{1}{3} \times 5 = 11.667$ m

$\therefore$ Residual head at A,

$$h_1 = 5\left(1 - \frac{11.667}{44.333}\right) = 3.684 \text{ m}$$

$\therefore$ Uplift pressure @ $A = 9.81 \times 3.684 = 36.142$ kN/m^2

Thickness of floor at point A,

$$t_1 = \frac{h_1}{G_C - 1} = \frac{3.684}{2.24 - 1} = 2.971$$

Increasing by 33%;

$t_1 = 1.33 \times 2.971 = 3.951$ m

b) **Uplift pressure and floor thickness at B,**

Creep length up to $B = (5 \times 2) + (3 \times 2) + \frac{1}{3} \times 10 = 19.333$ m

Residual head at B,

$$h_2 = 5\left(1 - \frac{19.333}{44.333}\right) = 2.82 \text{ m}$$

$\therefore$ Uplift pressure @ $B = 9.81 \times 2.82 = 27.664$ kN/m^2

Thickness of floor at B,

$$t_2 = \frac{h_2}{G_C - 1} = \frac{2.82}{2.24 - 1} = 2.274 \text{ m}$$

Increasing by 33%;

$$t_2 = 1.33 \times 2.274 = 3.024 \text{ m}$$

c) **Uplift pressure and floor thickness at C,**

Creep length up to $C = 5 \times 2 + 3 \times 2 + \frac{1}{3} \times 18 = 22$ m

Residual head at C,

$$h_3 = 5\left(1 - \frac{22}{44.333}\right) = 2.519 \text{ m}$$

∴ Uplift pressure @ $C = 9.81 \times 2.519 = 24.711 \text{ kN/m}^2$

Thickness of floor at C,

$$t_3 = \frac{h_3}{G_C - 1} = \frac{2.519}{2.24 - 1} = 2.031 \text{ m}$$

Increasing by 33%;

$$t_3 = 1.33 \times 2.031 = 2.701 \text{ m}$$

❖ Khosla's Theory and Concept of Flow Nets

- Many of the important hydraulic structures, such as weirs and barrage, were designed on the basis of Bligh's theory between the periods 1910 to 1925. In 1926 – 27, the upper *Chenab canal* siphons, designed on Bligh's theory, started posing undermining troubles. Investigations started, which ultimately lead to *Khosla's theory*. The main principles of this theory are summarized below:

- (a) The seepage water does not creep along the bottom contour of pucca floor as started by Bligh, but on the other hand, this water moves along a set of stream-lines. This steady seepage in a vertical plane for a homogeneous soil can be expressed by *Laplacian equation:*

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dz^2}$$

- Where, φ = Flow potential = Kh; K = the co-efficient of permeability of soil as defined by Darcy's law, and h is the residual head at any point within the soil.

- The above equation represents two sets of curves intersecting each other orthogonally. The resultant flow diagram showing both of the curves is called a *Flow Net.*

- *Stream lines*
- The streamlines represent the paths along which the water flows through the sub-soil. Every particle entering the soil at a given point upstream of the work, will trace out its own path and will represent a streamline. The first streamline follows the bottom contour of the works and is the same as Bligh's path of creep. The remaining streamlines follows smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown below.

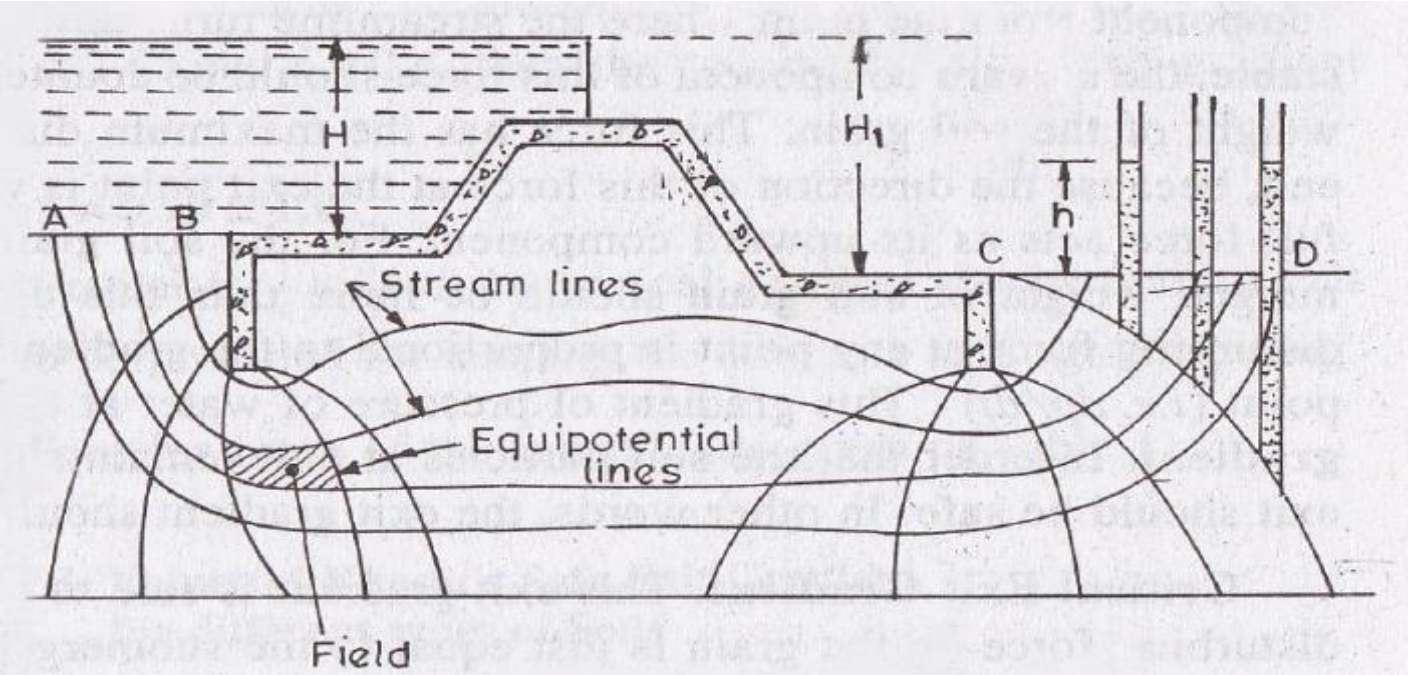


Fig: Khosla's Flow Net

- Equipotential lines

- Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily started that every streamline possesses a head equal to h1 while entering the soil; and when it emerges at the down-stream end into the atmosphere, its head is zero. Thus, the head h1 is entirely lost during the passage of water along the streamlines. Further, at every intermediate point in its path, there is certain residual head (h) still to be dissipated in the remaining length to be traversed to the downstream end. This fact is applicable to every streamline, and hence, there will be points on different streamlines having the same value of residual head h. If such points are joined together, the curve obtained is called an equipotential line.
- Every water particle on line AB is having a residual head h = h1, and on CD is having a residual head h = 0, and hence, AB and CD are equipotential lines.
- Since an equipotential line represent the joining of points of equal residual head, hence if piezometers were installed on an equipotential line, the water will rise in all of them up to the same level as shown in figure below.

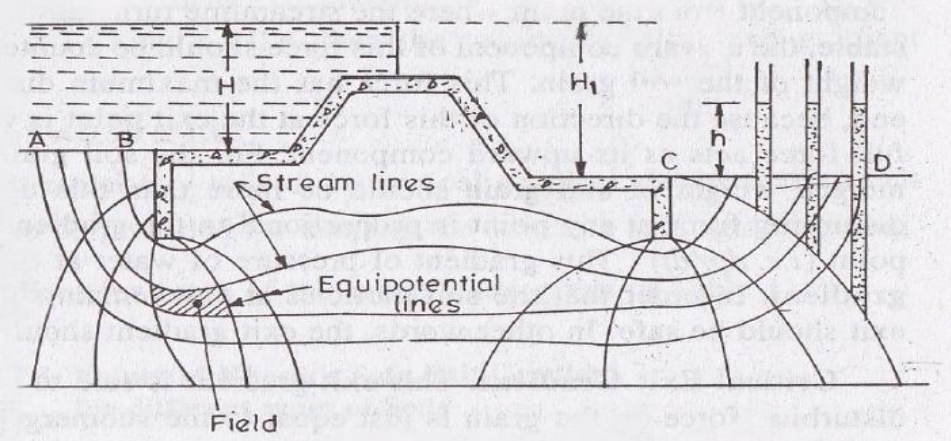
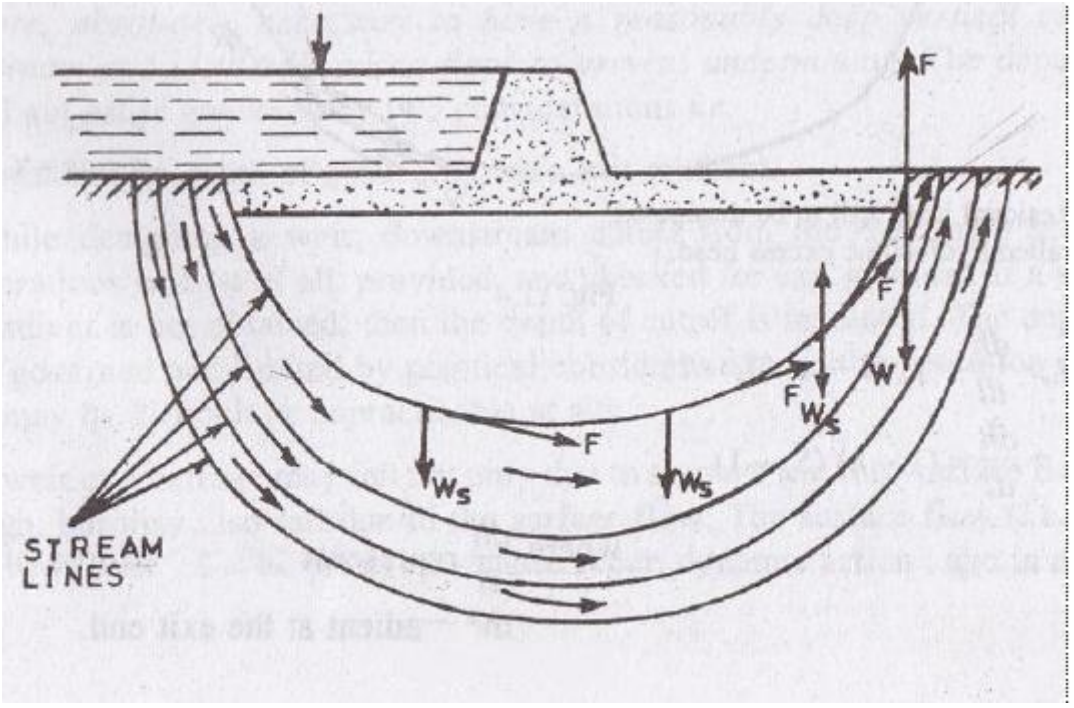


Fig: Khosla's Flow Net

- (b) The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in figure above. This force (F) has an upward component from the point where the streamlines turns upward. For soil grains to remain stable, the upward component of this force should be counterbalanced by the submerged weight of the soil grain. This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward, and hence full force acts as its upward component. For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward disturbing force. The disturbing force at any point is proportional to the gradient of pressure of water at that point (i.e. dp/dl). This gradient of pressure of water at the exit end is called the ***exit gradient.*** In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.



Critical exit gradient

- This exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. When a factor of safety equal to 4 to 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to ¼ to 1/5 of the critical exit gradient is ensured, so as to keep the structure safe against piping.

- The submerged weight (Ws) of a unit volume of soil is given as:
- $\gamma_w (1 - n) (S_s - 1)$

Where, γ_w = unit weight of water.

S_s = Specific gravity of soil particles

n = Porosity of the soil material

- For critical conditions to occur at the exit point

$F = W_s$

Where F is the upward disturbing force on the grain

Force F = pressure gradient at that point $= dp/dl = \gamma_w \times dh/dl$

Khosla's Conclusion

1. The outer face of the end sheet piles are more effective than the inner ones and the horizontal length of floor.
2. If intermediate sheet pile is smaller in length than the outer one then they were ineffective. (except for local redistribution of pressure)
3. It was essential to have deep vertical cutoff at the D/s end of the floor to prevent from piping.
4. Piping of floor started from the tail end, if the exit gradient at the D/s end is more than the critical exit gradient. The soil particle will move with the flow of water causing degradation of sub soil and resulting in failure.

For safety, exit gradient must be less than the critical exit gradient.

Considering factor of safety, safe exit gradient is taken as ¼ to 1/5 of critical exit gradient

❖ Khosla's method of independent variables (Calculation of percentage pressure, or uplift pressure by Khosla's theory

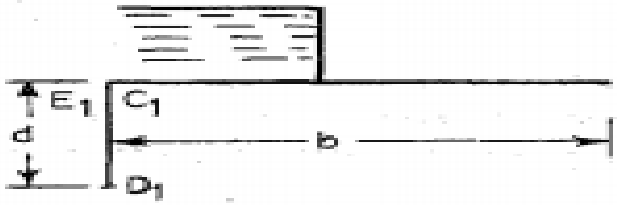
- Most designs do not confirm to elementary profiles (specific cases). In actual cases we may have a number of piles at upstream level, downstream level and intermediate points and the floor also has some thickness.
- Khosla solved the actual problem by an empirical method known as method of independent variables.
- This method consists of breaking up a complex profile into a number of simple profiles, each of which is independently amiable to mathematical treatment. Then apply corrections due to thickness of slope of floor.
- As an example the complex profile shown in fig is broken up to the following simple profile and the pressure at Key Points obtained.

(a)Straight floor of negligible thickness with pile at upstream ends.

(b)Straight floor of negligible thickness with pile at downstream end.

(c)Straight floor of negligible thickness with pile at intermediate points.

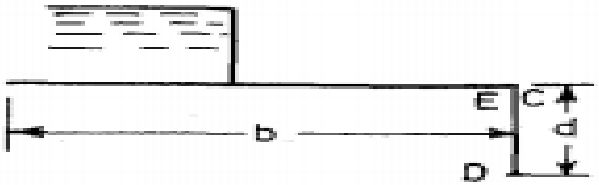
(d) A straight horizontal floor depressed below the bed but no vertical cut off.



$$\Phi_{C_1} = 100 - \Phi_E$$

$$\Phi_{D_1} = 100 - \Phi_D$$

(a)



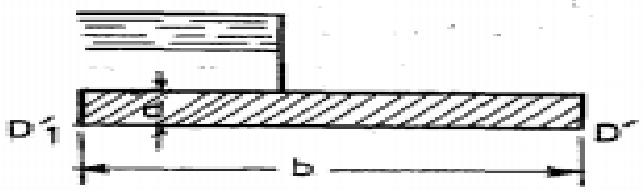
$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$

$\alpha = \dfrac{b}{d}$ (respective)

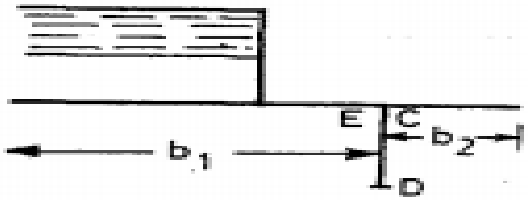
(b)



$$\Phi_{D'} = \frac{2}{3}(\Phi_E - \Phi_D) + \frac{3}{\alpha^2}$$

$$\Phi'_{D_1} = 100 - \Phi_{D'}$$

(c)



$$\Phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\Phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\Phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where $\lambda = \dfrac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$

$\lambda_1 = \dfrac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$

$\alpha_1 = b_1/d$

$\alpha_2 = b_2/d$

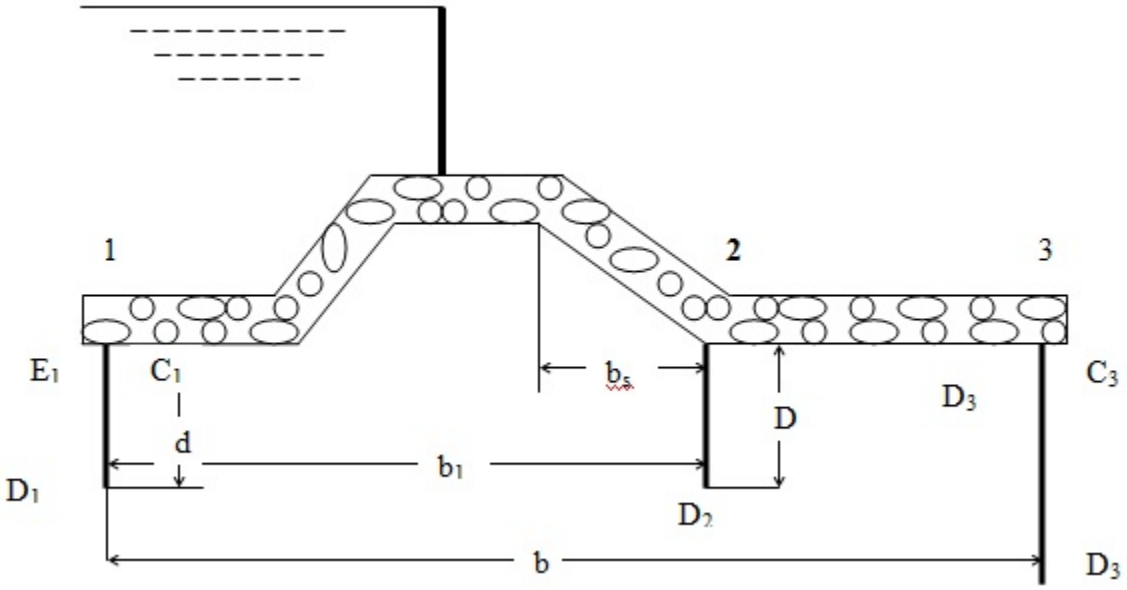
(d)

Fig. 11.5. Khosla's simple profiles for a weir of complex profile.

❖ Correction for percentage pressure

- The key points are the junctions of the floor and the pole lines on either side, and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

(a) Correction for the Mutual interference of Piles
(b) Correction for the thickness of floor
(c) Correction for the slope of the floor



(a) Correction for the Mutual interference of Piles

The correction C to be applied as percentage of head due to this effect, is given by

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

Where,

b' = The distance between two pile lines.

D = The depth of the pile line, the influence of which has to be determined on the neighboring pile of depth

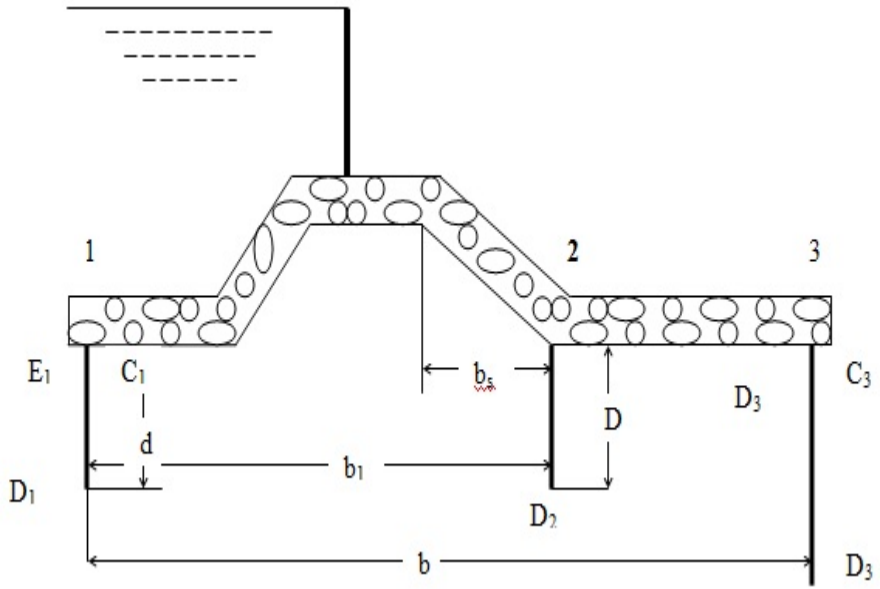
D is to be measured below the level at which interference is desired.

d = The depth of the pile on which the effect is considered

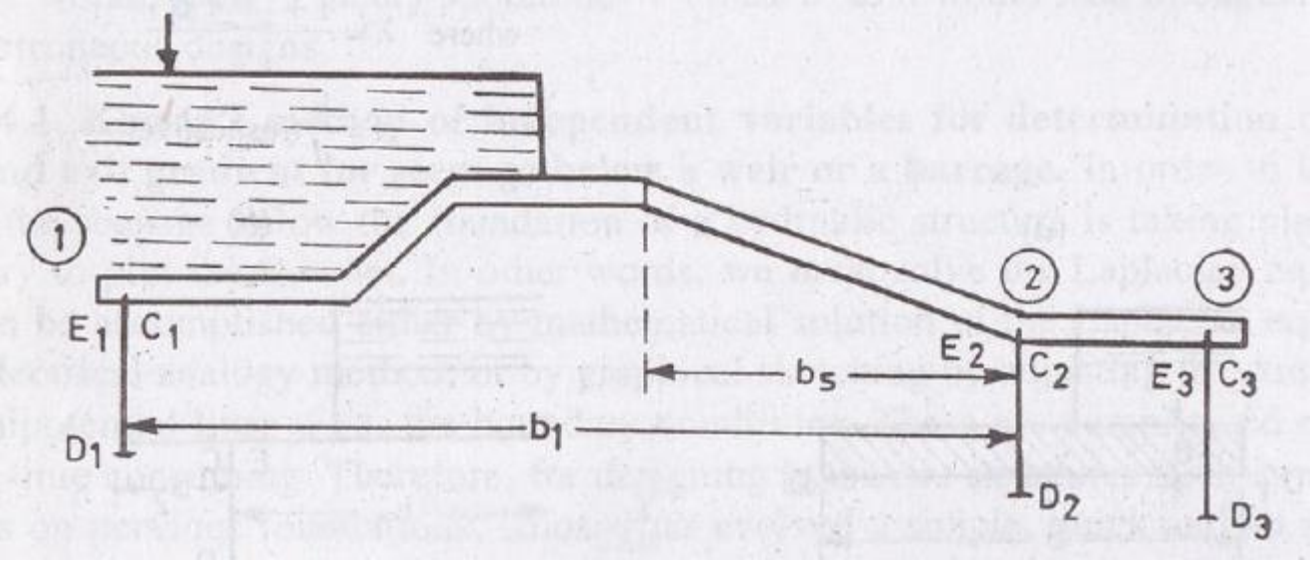
b = Total floor length

Sign Convention

- Positive for point in rear or backwater
- Negative for point on forward direction of flow



- The correction is positive for the points in the rear of back water, and subtractive for the points forward in the direction of flow. This equation does not apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is at a distance less than twice the length of the outer pile.

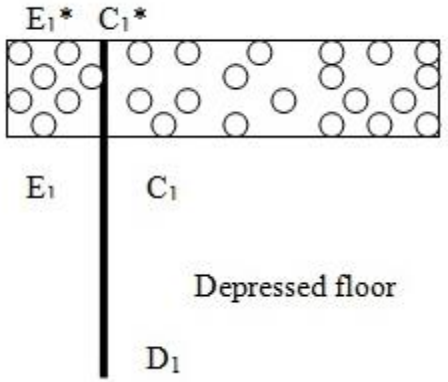


- Suppose in the above figure, we are considering the influence of the pile no (2) on pile no (1) for correcting the pressure at C1. Since the point C1 is in the rear, this correction shall be positive. While the correction to be applied to E2 due to pile no (1) shall be negative, since the point E2 is in the forward direction of flow. Similarly, the correction at C2 due to pile no (3) is positive and the correction at E2 due to pile no (2) is negative.

(b) Correction for thickness of floor

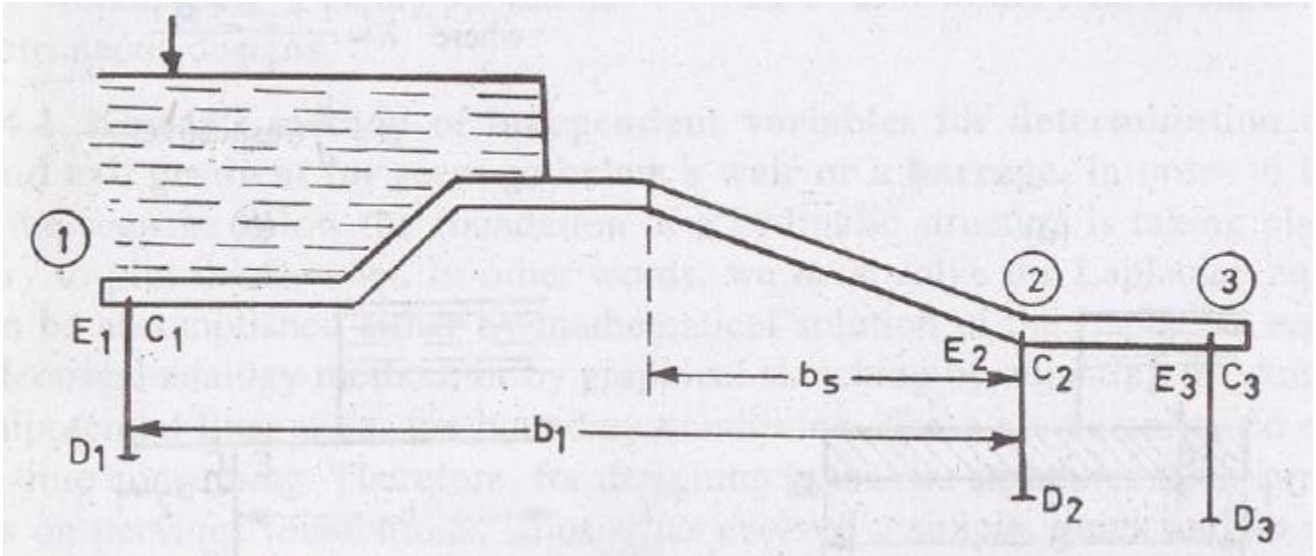
Thickness correction for C1 = $\frac{\phi D_1 - \phi C_1}{RL\ of\ floor\ Level - RL\ of D_1}$ * Thickness of floor at C_1

Thickness correction for E_2 = $\frac{\phi D_2 - \phi E_2}{RL\ of\ floor\ Level - RL\ of D_2}$ * Thickness of floor at E_2



Sign Convention

- Positive for point in the right side of the pile
- Negative for point in the left side of pile



(c) Correction for slope of the floor

It is given by,

$$C = C_S * \frac{b_S}{b_1}$$

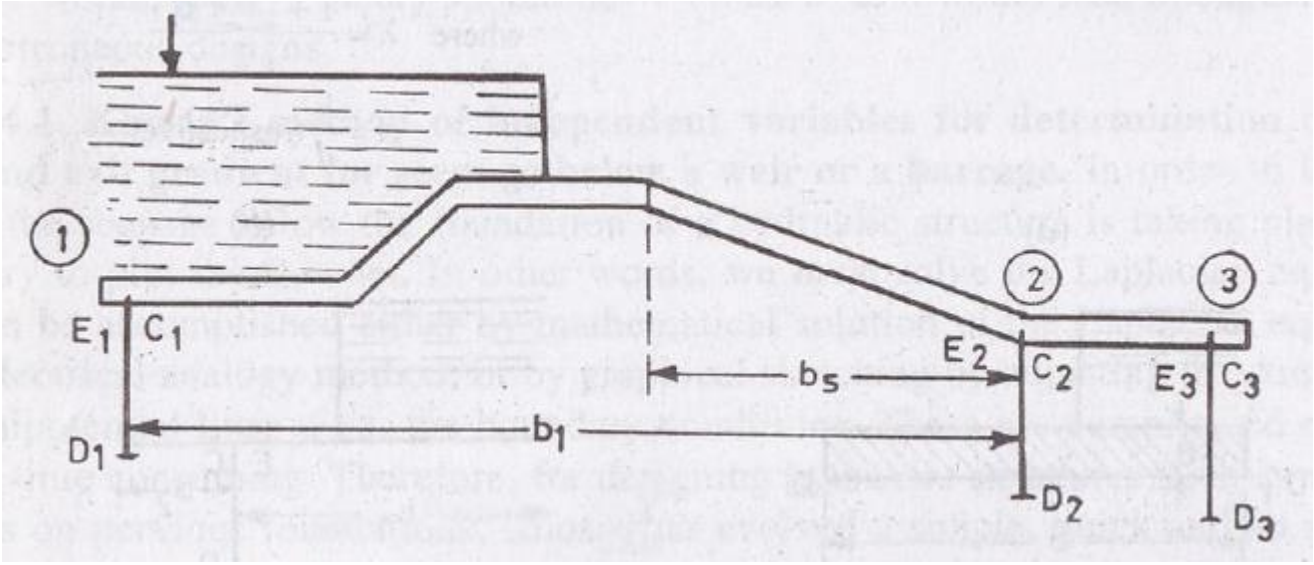
Where, b_S and b_1 are shown in figure

and C_S is slope correction factor and is given in table

Sign Convention

- Positive for the downward slope
- Negative for upward slope

Slope (H : V)	Correction Factor
1 : 1	11.2
2 : 1	6.5
3 : 1	4.5
4 : 1	3.3
5 : 1	2.8
6 : 1	2.5
7 : 1	2.3
8 : 1	2.0



Exit gradient

- It has been determined that for a standard form consisting of a floor length ‘b’ with vertical cutoff of depth ‘d’, exit gradient as it’s downstream end is given by;

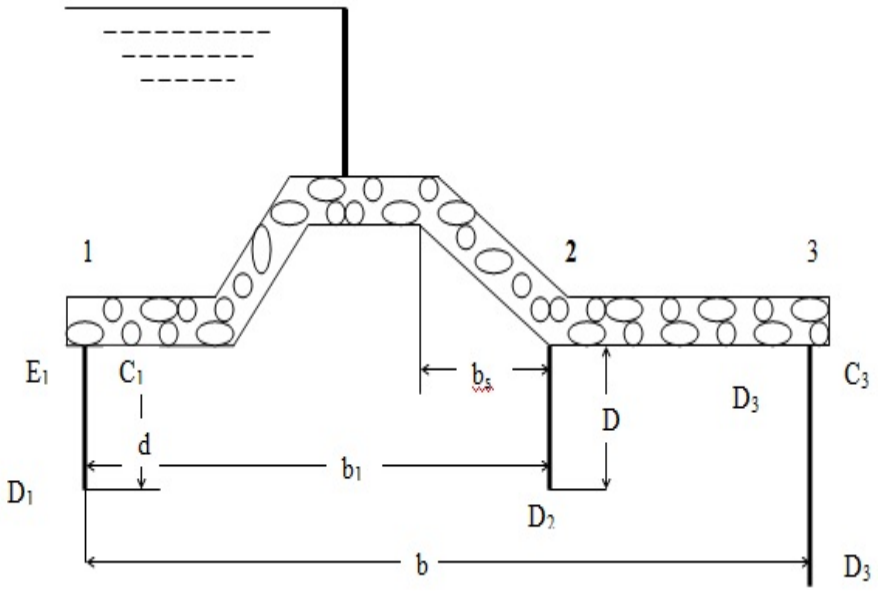
The exit gradient, $G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$

where $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$

and $\alpha = \frac{b}{d}$

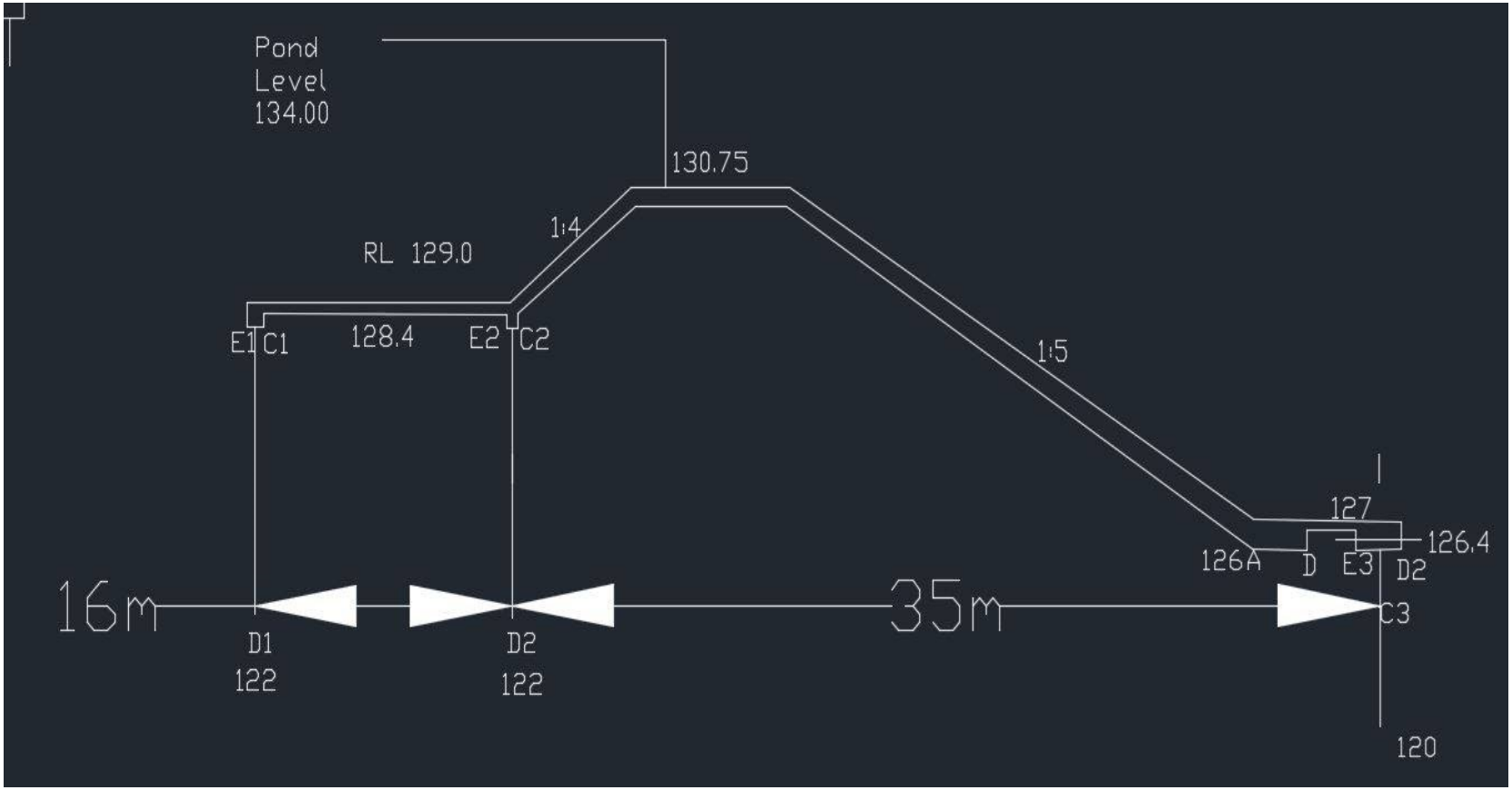
b is length of floor

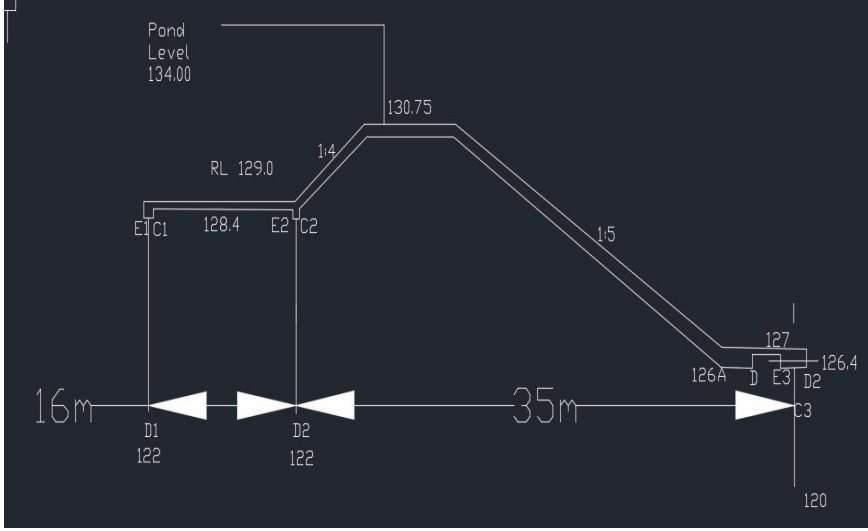
d is depth of vertical cut off



Type of Soil	Safe exit gradient
Shingle	1/4 to 1/5 (0.25 to 0.20)
Coarse Sand	1/5 to 1/6 (0.20 to 0.17)
Fine Sand	1/6 to 1/7 (0.17 to 0.14)

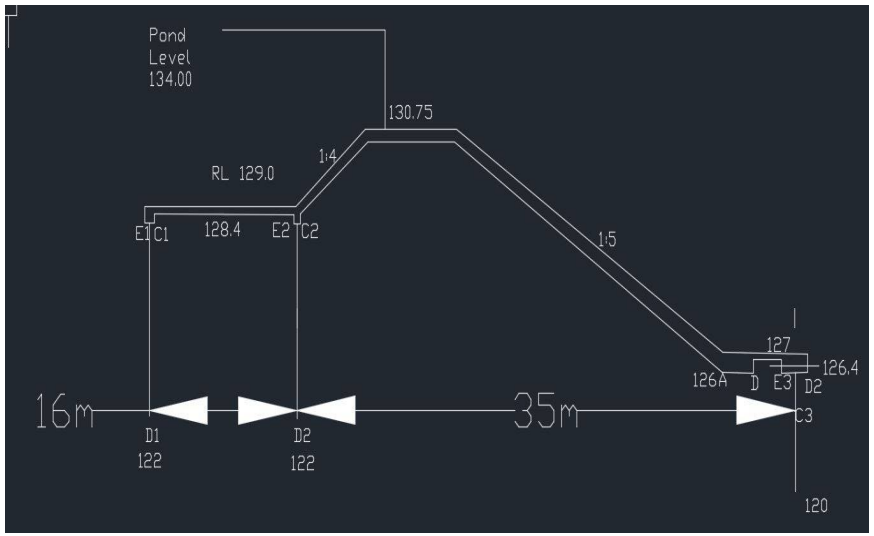
Q.N. Using Khosla's method, check the safety of weir profile shown below against piping and uplift (at point 'A'). Safe exit gradient may be assumed to be 1 in 5.



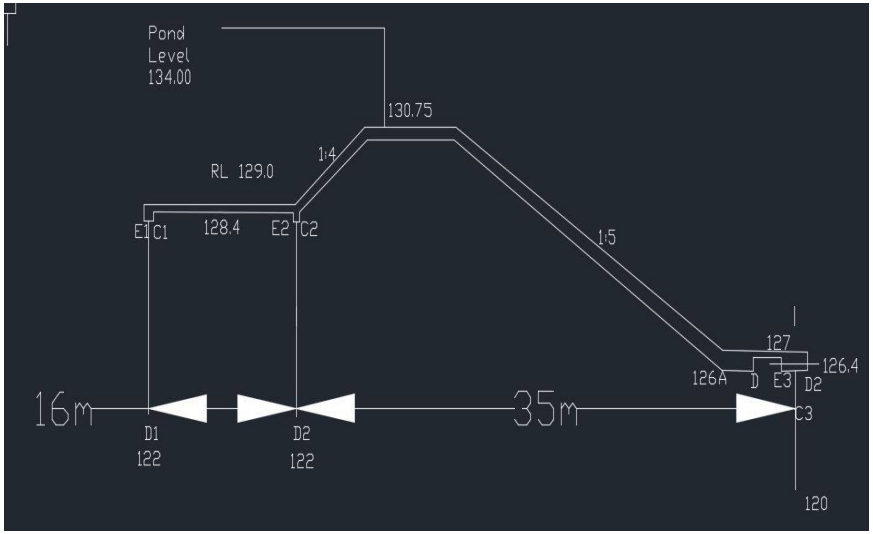


- For u/s pile number 1
- b= 51m
- d= 129-122=7m
- α=51/7 = 7.286
- $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ =4.18
- $\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)=\frac{1}{\pi}\cos^{-1}\left(\frac{4.18-2}{4.18}\right) * \frac{\pi}{180}$=0.33 = 33%
- ϕ_{c_1}=100-ϕ_E=100-33=67%
- We have
- ϕ_{D_1}=100-ϕ_D
- $\phi_D=\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)=\frac{1}{\pi}\cos^{-1}\left(\frac{4.18-1}{4.18}\right) * \frac{\pi}{180}$=0.22=22%
- ϕ_{D_1}=100-22=78%
- Correction for c_1
- i)Correction due to mutual interference of pile 2
- D=128.4-122=6.4m
- d'= 128.4-122=6.4m
- b' =16m
- b =51m
- C =19$\sqrt{\frac{6\cdot4}{16}}\frac{6.4+6.4}{51}$=3.02%(Positive)

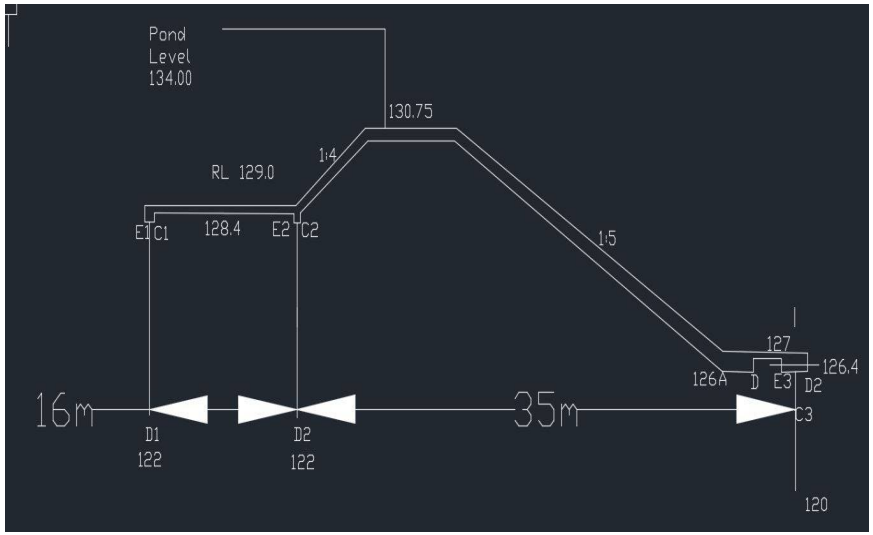
- ii)Correction due to to thickness of floor
- C =$\frac{78-67}{129-122}$*0.6=0.94%(positive)
- iii)Correction due to slope is zero
- corrected ϕ_{c_1}=67+3.02+0.94=70.96%
- For intermediate pile number 2
- d=129-122=7m
- b_1=16m
- b2=35m
- α_1=16/7=2.29
- α_2=35/7=5
- $\lambda = \frac{\sqrt{1+\alpha^2}_1+\sqrt{1+\alpha^2}_2}{2}$=3.8
- $\lambda_1 = \frac{\sqrt{1+\alpha^2}_1-\sqrt{1+\alpha^2}_2}{2}$=-1.3
- $\phi_{E_2}=\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right)$=0.71=71%
- $\phi_{D_2}=\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$=0.61=61%
- $\phi_{c_2}=\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right)$=0.53=53%
- Correction for ϕ_{E_2}



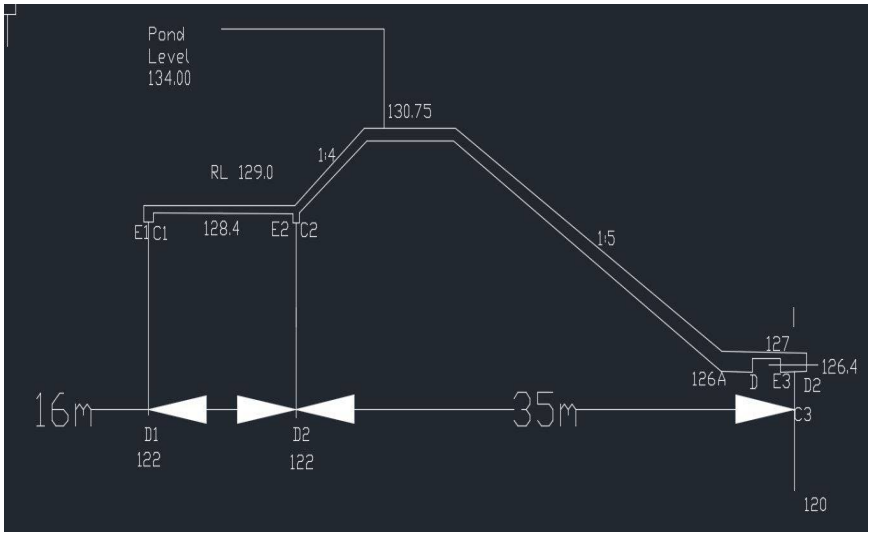
- i) correction of E_2 due to mutual interference of pile 1
- $C = 19\sqrt{\frac{D}{b'}}\frac{d+D}{b}$
- D=128.4-122=6.4m
- d=128.4-122=6.4m
- b' =16m
- b= 51m
- C=3.02% (Negative)
- ii) Correction due to floor thickness
- C = $\frac{71-61}{129-122} \times 0.6 = 0.86\%(negative)$
- iii) Slope correction is nil
- corrected ϕ_{E_2}=71-3.02-0.86=67.12%
- Correction for ϕ_{c_2}
- i)Mutual interference due to pile 3
- D=128.4-120=8.4m
- d=128.4-122=6.4m
- b'=35m
- b=51m
- C=2.7%(Positive)



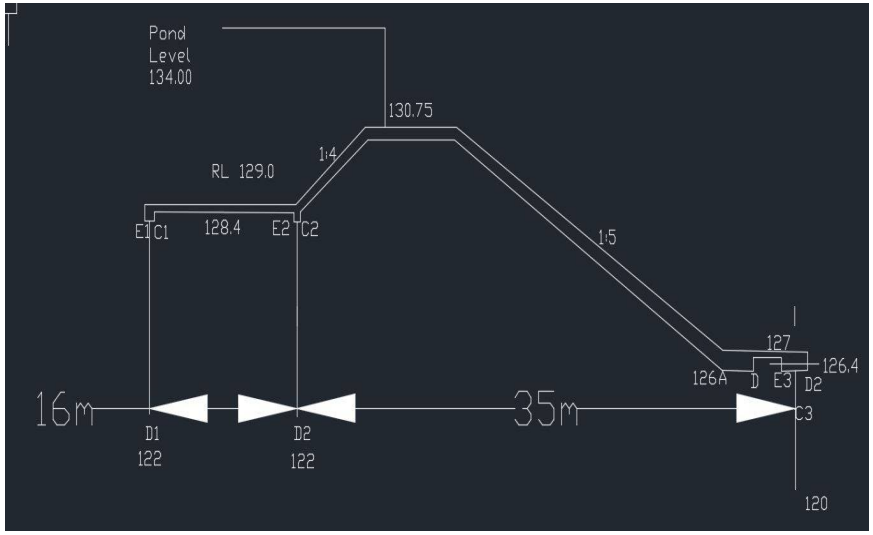
- ii)Due to floor thickness
- C=$\frac{61-53}{129-122} \times 0.6 = 0.69\%(Positive)$
- iii)Correction due to slope
- C=$C_5 \times \frac{b_s}{b_1}$
- bs=1.75*4=7m
- b_1=35m
- C_5=3.3 for 4:1 referring table of slope correction factor
- C=0.66% (Negative for upward slope)
- Corrected ϕ_{c_2}=53+2.7+0.69-0.66=55.73%
- For d/s pile number 3
- b=51m
- d=127-120=7m
- α =b/d=7.29m
- $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ =4.18
- ϕE_3=$\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) * \frac{\pi}{180}$=0.33=33%
- ϕ_{D_3}=$\frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$=0.22 =22%
- Correction for ϕ_{E_3}



- i)Mutual interference due to pile 2
- D=126.4-122=4.4m
- d=126.4-120=6.4m
- b'=35m
- b=51m
- C=1.426% (Negative)
- ii)Thickness correction
- C=$\frac{33-22}{127-120} \times 0.6 = 0.94\%$ $(Negative)$
- iii)Slope correction is nil
- corrected ϕ_{E_3}=33-1.426-0.94=30.624%
- Difference between u/s water level and d/s water level = 134 – 127 = 7 m
- Uplift pressure @C_1=0.7096*7 = 4.9 m
- @D_1 = 0.78*7 = 5.46 m
- @E_2 = 0.6712*7 = 4.7 m
- @D_2 = 0.61*7 = 4.27 m
- @C_2 = 0.5573*7 = 3.9 m
- @E_3 = 0.30624*7 = 2.144 m
- @D_3 =0.22*7 = 1.54 m
- Uplift pressure at A (h) = 2.144 + $\frac{3.9-2.144}{35}$*(35-7-2-18.75) = 2.507 m



- Assuming,
- Crest width =2m
- h=2.507m
- Thickness required at A=$\frac{h}{G_c-1}$=$\frac{2.507}{2\cdot24-1}$= 2.021m>1m(unsafe)
- Exit gradient
- We have,
- Exit gradient (G_E)=$\frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$
- H = maximum seepage head=134-127=7m
- d= depth of d/s cutoff =127-120=7m
- $\alpha = \frac{b}{d}$=51/7=7.29m
- $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$=4.18
- G_E)= $\frac{7}{d7}\frac{1}{\pi\sqrt{4.18}}$=0.16<1/5(safe)
- The exit gradient is equal to 0.16 I.e, 1 in 6.42 which is safe.



❖ Design of silt excluder

I. Silt excluder are designed for 15 to 20 % of canal full supply discharge.

II. A minimum velocity of 2 to 4.5 m/s should be maintained in order to eliminate silt deposition. Generally, 2 m/s for sandy rivers, 4 to 4.5 m/s for boulder stage rivers and 3 m/s for ordinary straight reaches.

III. Knowing the discharge and velocity the cross section area of tunnel opening can be determined. $A = Q/V$

IV. The height of the tunnel (h) is equal to the height of the crest of head regulator above the u/s floor of undersluices minus thickness of roof slab.

V. Knowing the height (h), width is obtained as $B = A/h$

VI. Total clear width is divided into number of tunnel such that the span is 1.8 m to 2.4 m for each tunnel separated by a divide wall of 0.6 m.

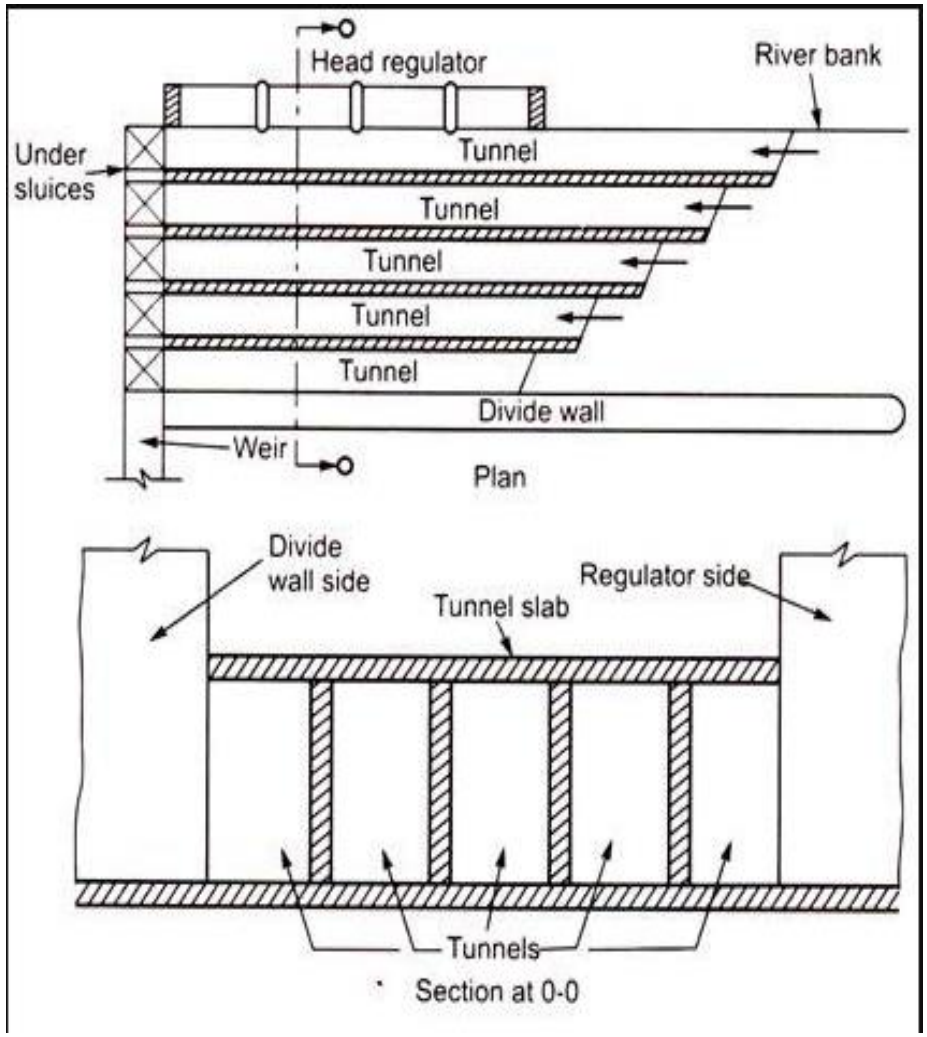
VII. Generally 4 to 6 tunnel are provided for each excluder.

VIII. The length of tunnel is different but head loss in each tunnel should be kept equal.

IX. The height of tunnel varies from 0.5 to 0.6 m for sandy rivers and 0.8 to 1.2 m for boulder stage rivers.

X. At entrance. The tunnel are generally given a bell mouth shape and radius of bell mouthing generally varies from 2 to 6 time the tunnel width.

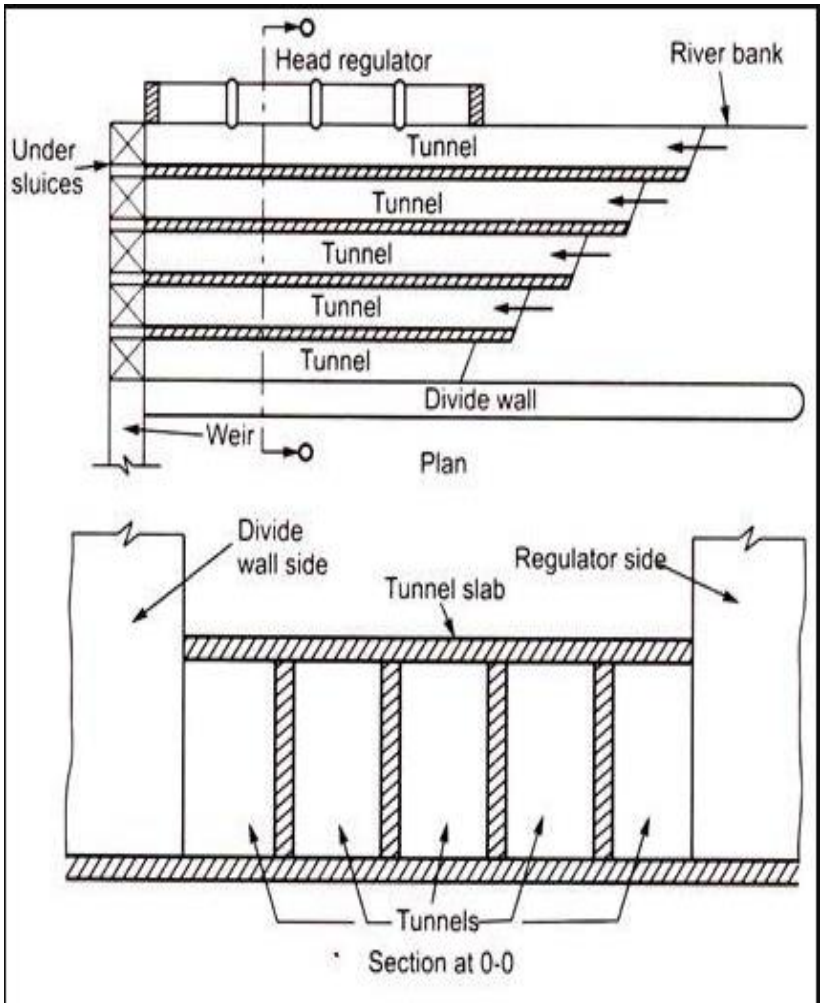
XI. At the exit, the tunnel are throttled for restricting the discharge to the desired value and to increase velocity to prevent deposition of silt.


Head regulator
River bank
Under sluices
Tunnel
Tunnel
Tunnel
Tunnel
Tunnel
Divide wall
Weir
Plan
Divide wall side
Regulator side
Tunnel slab
Tunnels
Section at 0-0

Q.N. Design the silt excluder for the diversion headwork for the data given below:
Full supply discharge of canal = 200 cumecs
Crest level of undersluices = 200 m
Crest level of head regulator = 202.00 m
Bay width of undersluices = 16 m

- Solution:

Let the design discharge of silt excluder
be 20 % of that of canal design discharge
Q = 0.2*200 = 40 m3/s
Let us assume the velocity of 2 m/s, then,
Area of X-Section (A) = Q/V = 40/2 = 20 m2
Let thickness of roof slab = 0.2 m
Height of tunnel = 202.00 – 0.2 -200 = 1.8 m
Total clear width = 20/1.8 = 11.11 m
For clear span of 2.1 m,
Number of tunnels = 11.11/2.1 = 5.29 (say 6.00 m)
let., thickness of divide wall = 0.65 m
Overall width = 6*2.1+5*0.65
= 15.85 (says 16 m)



❖ Silt ejector or silt extractor

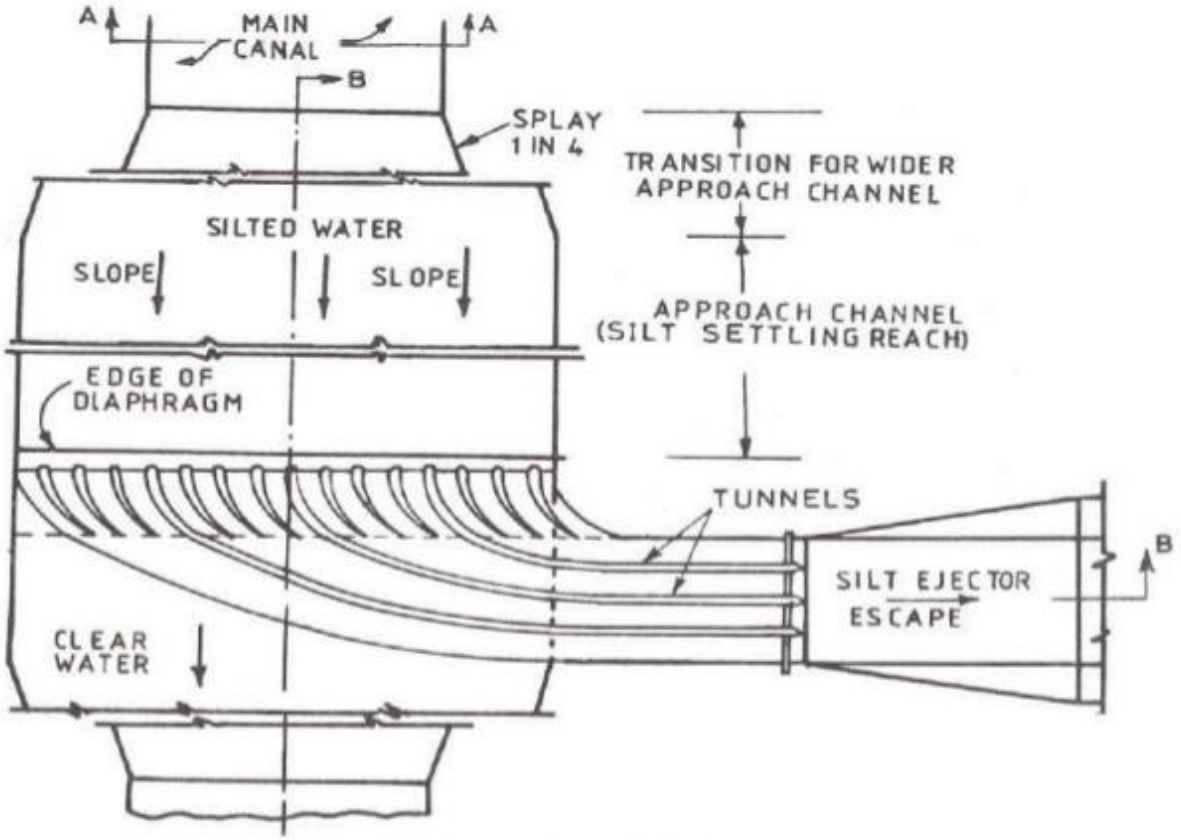


Fig: Plan of Silt Ejector

❖ Design of slit ejector

- Silt ejectors are designed for 20 to 25 % of the canal discharge.
- Bed of the canal is depressed by 0.3 m to 0.5 m at the mouth of the ejector so that approach velocities are reduced and the bed load may be trapped.
- The tunnel are of low height, about 0.5 to 0.6 m.
- The ideal distances between head regulator and silt ejector is usually between 150 m to 500 m.
- The section of the sub tunnel is gradually reduced such that there is an overall increase of 10 to 15 % in the velocity up to the exit.
- The section of the main tunnel at exit is usually designed to attain the velocity of 2.5 to 6 m/s depending upon the grade of the sediment to be removed.
- The portion of the approach channel immediately upstream of the ejector should be pitched for a length of 3 to 4 times the depth of the water in the channel so that there is no erosion of the bed and sides of the channel.

Design glacis weir for the following data,

- **Maximum discharge intensity on weir crest = 12.5 cumecs/m**
- **H.F.L. before construction of weir = 225.00 m**
- **River bed level = 218.75 m**
- **Pond level = 224.00 m**
- **Height of crest shutters = 1.5 m**
- **Anticipated d/s water level in the river when weir is discharge with pond level u/s = 221.5 m**
- **Bed retrogression = 0.5 m**
- **Lacey's silt factor = 0.9 m**
- **Permissible exit gradient = $\frac{1}{7}$**
- **Permissible afflux = 1 m**

Step 1: Crest level

- Pond level = 224.00 m
- Height of shutter = 1.5 m
- Crest level = 224.00 – 1.50 = 222.50 m

Step 2: u/s T.E.L.

- q = 1.7 $K^{\frac{3}{2}}$
- K = $\left(\frac{q}{1.7}\right)^{\frac{2}{3}}$ = 3.78 m
- Level of u/s T.E.L. = Crest level + K
- = 222.50 + 3.78 = 226.28 m

step 3: scour depth, velocity and velocity head

- regime scour depth,
- R = 1.35 $\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$
- = 7.5 m
- Regime velocity (V) = $\frac{q}{R}$
- = 1.67 m/sec
- Velocity head = 0.14 m.

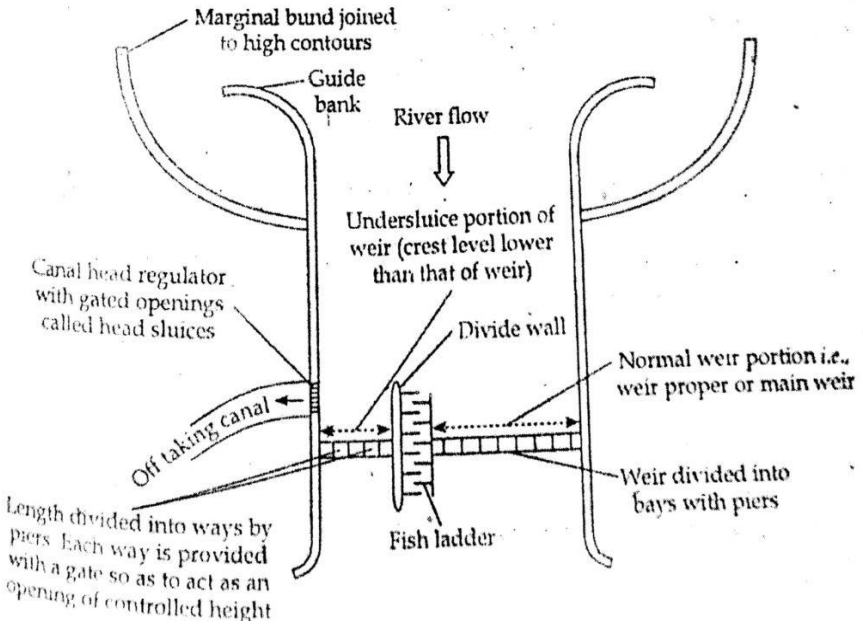
step 4: High flood condition

- u/s H.F.L. = u/s T.E.L. – velocity head
- = 226.28 – 0.14
- = 226.14
- d/s T.E.L. = HFL before construction + velocity head = 225.00 + 0.14 = 225.14 m
- Afflux = u/s HFL – d/s HFL = 226.14– 225.00 = 1.14 m
- Which is very near to the permissible afflux of 1 m
- The d/s HFL after retrogression = HFL before construction – 0.50
- = 225.00 – 0.5 = 224.5 m
- d/s T.E.L. after retrogression = 225.14 - 0.5 = 224.64 m
- loss of head at high flood level = u/s T.E.L. – d/s T.E.L. after retrogression
- = 226.28 – 224.64 = 1.64 m

Design of Undersluices

Design procedure

- Fix the discharge over the weir bay and undersluice bay section. i.e, 20 % over the sluice section and 80 % over the weir bay section.
- Fix the crest levels of undersluice section and the weir bay section.
- Crest level of undersluice = Deepest bed level
- The slope of the d/s glacis is usually kept between 3:1 and 5:1.
- Crest level of weir bay = Crest level of undersluices + (1 to 1.5 m).
- Fix the waterways for the weir bay and undersluices sections.


Marginal bund joined to high contours
Guide bank
River flow
Undersluice portion of weir (crest level lower than that of weir)
Canal head regulator with gated openings called head sluices
Divide wall
Normal weir portion i.e., weir proper or main weir
Off taking canal
Weir divided into bays with piers
Length divided into ways by piers. Each way is provided with a gate so as to act as an opening of controlled height
Fish ladder

Design the under sluice section of diversion headworks with the following data.

- ✓ **Design flood discharge in the river = 9000 cumecs.**
- ✓ **Deepest bed level of the river = 200.00 m**
- ✓ **HFL before construction = 206.00 m**
- ✓ **FSL of canal = 203.00 m**

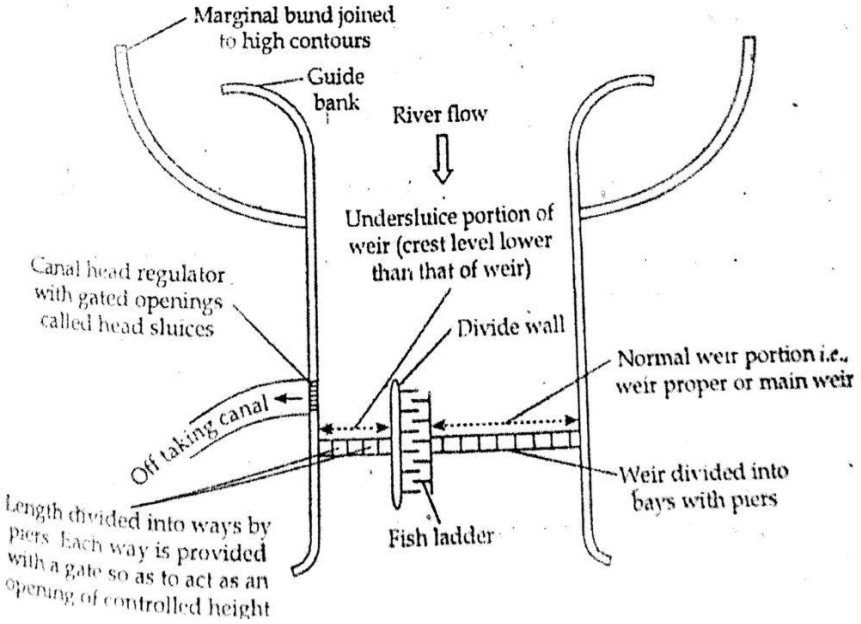
Solution:

- Crest level of undersluices = Deepest bed level = 200.00 m
- Crest level of weir bay = 200 + 1.5 = 201.5 m
- Pond level = FSL +Modular head = 203 +1 = 204.00 m (Modular head is equal to the sum of head loss in the regulator and the head required to pass full supply discharge into the canal. It is generally taken as 1 m.)

Waterway

- From lacey's formula,
- P = 4.75 $\sqrt{Q}$ = 4.75 $\sqrt{9000}$ = 450.62 m

- Let us fix the water way as follows;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Off taking canal
Weir divided into bays with piers
Length divided into ways by piers. Each way is provided with a gate so as to act as an opening of controlled height
Fish ladder

Undersluices portion

- Number of spans = 5
- 5 bays of 16 m each = 80 m
- 4 piers of 2.5 m each = 10 m
- Total = 90 m

Weir bay portion

- Number of spans = 26
- 26 bays of 12 m each = 312 m
- 25 piers of 2 m each = 50 m
- Total = 362 m
- Width of fish ladder = 5 m
- Width of divide wall = 3 m
- Overall waterway between abutments = 90 + 362 + 5 + 3 = 460 m